THE UNIVERSITY OF ALBERTA

ELASTIC INSTABILITY OF A PONY-TRUSS BRIDGE

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

by

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FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommended to the Faculty of Graduate Studies for acceptance, a thesis entitled Elastic Instability of a Pony-Truss Bridge submitted by Frank Lukawitski, B.Sc. in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

The author presents the experimental procedure used in and the results obtained from a load test to failure of a pony-truss bridge. From the observed behavior of the structure during the load test, the author also endeavors to predict the failure load with available theories.

The maximum live load that the structure could carry during the test was forty-two tons. This load consisted of two point loads applied by means of hydraulic jacks which were symmetrically located six feet apart on the centre floor beam of the bridge.

The available theories predict values of ultimate load which indicate that forty-two tons is somewhat low. This possibility is illustrated by one of the theories which makes use of the test data in which the expected failure load of the north truss is greater than that of the south truss. This expectation is within reason since the compression chord of the south truss had large, initial, lateral deflections as compared to that of the north truss.

The energy method appears promising for predicting the ultimate capacity of the bridge. However, for the particular structure and the particular loading arrangement, the compression chord also acts as a beam with undetermined support conditions,

and thus the theoretical failure load could only be obtained with some aid from the observed behavior of the bridge.

A better correlation between the theoretical and test results would be preferred. This appears possible if a test is carried out in such a manner that the beam action of the compression chord is eliminated and also if the structure tested is constructed of straight members throughout.

ACKNOWLEDGMENTS

The author wishes to express his appreciation to Prof. J.S. Kennedy and Dr. G. Ford, Department of Mechanical Engineering, University of Alberta, for their continued help, generous contributions and criticisms towards the successful carrying out of the test, and towards the preparation of this thesis.

The author also wishes to express his thanks to the Research Council of Alberta and to the Bridge Branch of the Department of Highways of Alberta for sponsoring this project, and to Mr. B.P. Shields for his administration of the project and to Fred Vaneldik and everybody else who helped out with the preparation for, and the completion of, the test.

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CHAPTER I

INTRODUCTION

A. NATURE OF INVESTIGATION

The pony-truss bridge differs from an ordinary steel truss bridge in that it has no lateral ties between the panel points of the two top chords. This type of bridge serves its purpose economically where relatively short spans are required. For such short spans the required, economical height or depth of truss is not sufficient to allow overhead lateral ties between the two trusses.

Because of recent failures of such structures, the Department of Highways of Alberta initiated this test program with the intention of establishing the cause of failure. Although it was realized that the most probable cause of failure is the lateral buckling of the compression chord, the investigation was so conducted that the overall behavior of the structure could be observed to detect other factors that may cause or contribute to the failure.

The compression chord (top chord) of a pony truss behaves like an axially loaded column which is elastically supported in a horizontal plane by the vertical web members of the truss. The web verticals along with the floor members constitute a rigid frame shown in end view in Fig. 5. Due to the flexibility of this rigid frame the panel points (points of chord attached to the frame) cannot be considered as fixed. Under

such conditions, it is difficult to establish a mode of failure, because during the buckling of the top chord, shearing forces are set up which displace the points of support. These displacements are the same magnitude as the chord deflections. Thus the top chord will buckle in a different pattern for different values of the spring constants of the elastic supports. These different modes of buckling may be best explained by considering the two limiting modes of failure. If the supports are sufficiently stiff, the top chord will be forced to buckle in half-waves of length equal to the distance between two adjacent panel points. This case would represent the greatest axial load capacity of the top chord. On the other hand, if the elastic supports are relatively flexible, it is possible that the entire top chord will buckle in one-half wave. This limiting case would represent the lowest axial load capacity of the top chord.

From the foregoing statements it may appear that the whole problem may be solved by increasing the stiffness of the rigid frame to produce the first limiting case. This may be so for a condition where all the floor beams carry equal live loads. However, if the live load is carried on only a few floor beams, the deflection of the load-carrying floor beams will either introduce lateral forces on the top chord or reduce the magnitude

of the lateral support by decreasing the effective deflection of the elastic support during buckling.

B. HISTORICAL REVIEW

The subject of lateral stability of the unbraced chords of trusses such as the compression chords of pony-truss bridges is not new to the structural engineer. Since the late 1800's this problem has been recognized and much theoretical and some experimental work has been done to help predict the capacity and to understand the behavior of similar structures. Figure 1. is a photograph of one of the early failures of a pony-truss bridge.

The first to investigate the problem of a column supported by elastic supports was Engesser[1*]. He derived an approximate formula for the required stiffness of the elastic supports. Soon after, Jasinsky[2] extended this work by considering the case of axial loads varying along the column.

Following this, there are many other investigations which deal with the buckling of columns on elastic foundations. However, many of them either deal with some particular aspect of the problem or are too involved to be applied to the analysis of any individual, practical case. The investigations and theories that are most suited to this problem are listed herein, but it

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* Numbers refer to the references as presented in the Bibliography.

is necessary to point out that, in many cases, these are a result of the contributions of others which have not been mentioned.

In 1930, Timoshenko[3] published a paper in which he employed the energy method for solving these problems. This method lends itself to the analysis of more involved problems, such as those involving columns of varying rigidity, varying degrees of axial thrust along the column and a variation of the elastic stiffness of the supports. This theory appears to be suitable for determining the failure load of a pony-truss bridge, if the failure is a result of the lateral buckling of the top chord. This strain energy principle may be simply stated as the equating of the external work on the compression members with the internal work of the compression members and the lateral supports during the process of buckling.

The external work may be defined as the axial load on the column displaced by the axial deflection produced by buckling. The internal work is composed of the bending of the column and the deflection of the elastic supports.

The general assumptions on which this theory is based are:

1. The loads are applied in the plane of the truss on the lower panel points of the truss.

2. The dead loads of the structure are concentrated at the lower panel points.

3. The members are initially straight.

A full calculation using the strain energy equation is given in Appendix I.

Hrenikoff[4] published a paper on the "Elastic Stability of a Pony Truss" in 1935. He also used the energy method, but he was mainly concerned with the effect on the buckling of the top chord produced by the torsional bending of the top chord and the twisting of the verticals. One significant conclusion presented by Hrenikoff is that the mode of buckling of the top chord is not greatly affected by different loading conditions. He concludes that the results from one load test will give the buckling mode of the structure for any loading. Therefore the problem involving moving loads need not be considered.

More recently Bleich[5] discusses most of the earlier work and offers simplifying assumptions which may be considered when making use of the strain energy equation. He suggests that the axially loaded column on elastic supports may include the end diagonals as well as the top chord, and that the ends of the column may be assumed as pin-connected with the effective length, of that portion of the column from the pinned-end to the first panel point, equal to the horizontal projection of the end diagonal. It is interesting to note that Bleich is of the opinion that different

loading conditions have an effect on the failure load.

Other works which apply to the elastic instability of compression chords on elastic supports, which are not a direct mathematical approach, are those presented by Southwell[6] and by Shanley[7].

Southwell states that if for every known axial load (P) the maximum deflection (Δ) of a column is measured, then by plotting Δ/P vs. Δ the slope of the best fitting straight line is $1/P_{cr}$, the reciprocal of the critical load. This theory has been set up for the case of a simple column, and thus for other cases, Δ must be the maximum deflection of the column from the straight line joining the two adjacent points of inflection.

Shanley makes use of test data for determining the capacity of axially loaded members, by plotting the strains across a section of a column for every increment of load. At some stage of loading the bending strains increase so rapidly compared to the axial strains that the elements on the convex side of the member reverse their direction of strain. This cross-over of strains is the first indication of buckling, and may be taken as a very conservative value of buckling load.

C. SCOPE OF PRESENT INVESTIGATION

A used, eighty foot span, pony-truss bridge, (see Figs. 3 and 4) assembled in Lambton Park on the outskirts of Edmonton,

Alberta, was tested to obtain:

1. The ultimate load capacity of the bridge;

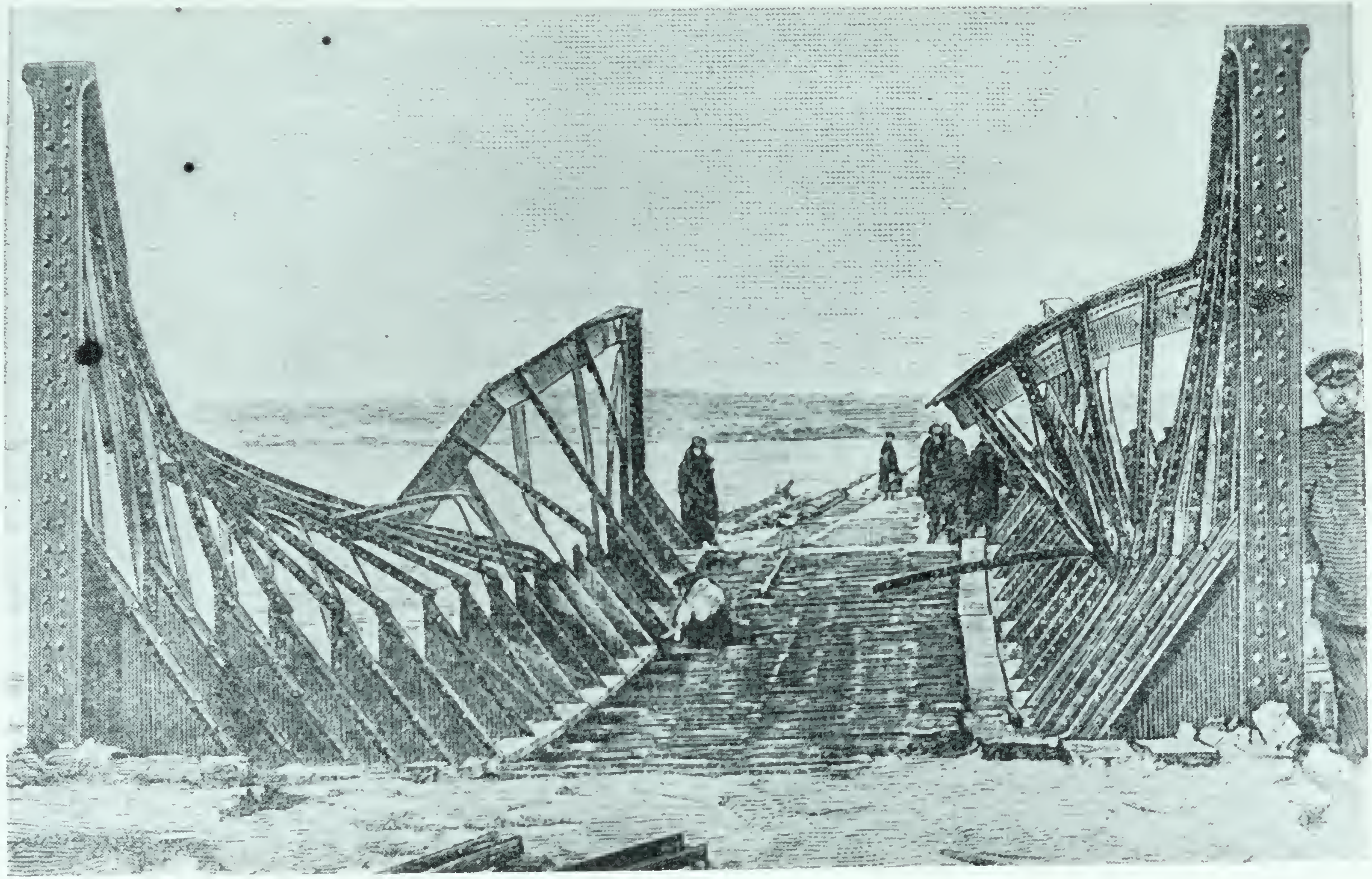
2. Sufficient information to make a comparison of actual failure load with that predicted by available theories;

3. The effect of bent members on the ultimate capacity of the bridge.

The third part of the problem arose because a used bridge was tested and several members in the bridge were bent. The member that caused the most concern was the top chord in the south truss which had an initial eccentricity of one inch from a line joining two adjacent panel points, (U_1-U_2) (See Figs. 2 and 3).

The original intent was to obtain the failure load of the used bridge, then rebuild the bridge with new straight members and again obtain the failure load. In this manner a comparison would be obtained between the capacity of the bridge with straight members and that of the used bridge with the bent members. To date, only the test on the used bridge has been completed, and the results presented are based on this test.

The author has been unable to discover any reports of similar investigations. In fact, aside from theoretical research and some testing of bars with elastic supports, to the author's knowledge, this if the first such test to be carried out.



By permission from HISTORY OF STRENGTH OF MATERIALS by S.P. Timoshenko. Copyright, 1952. McGraw-Hill Book Company.

Fig. 1

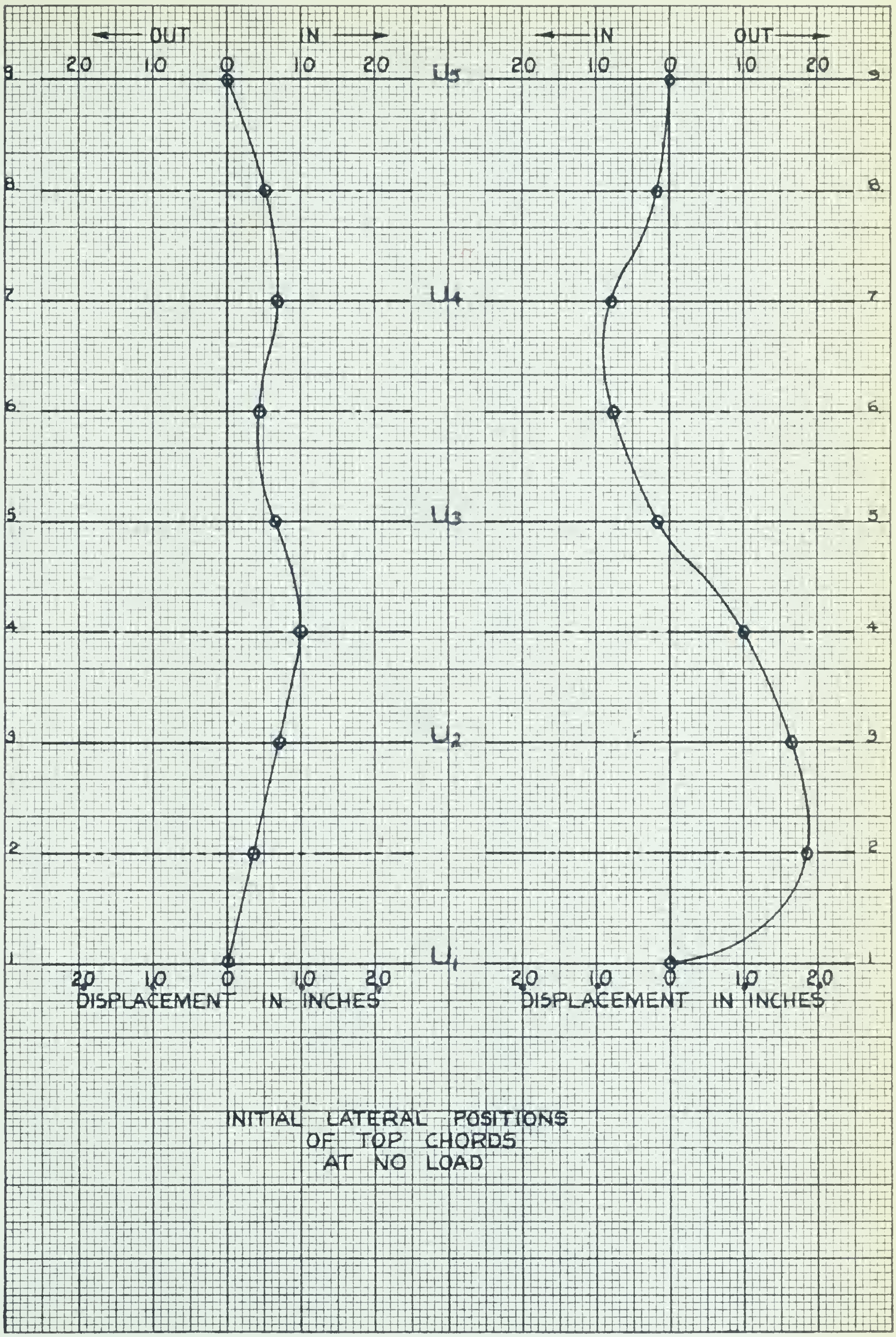
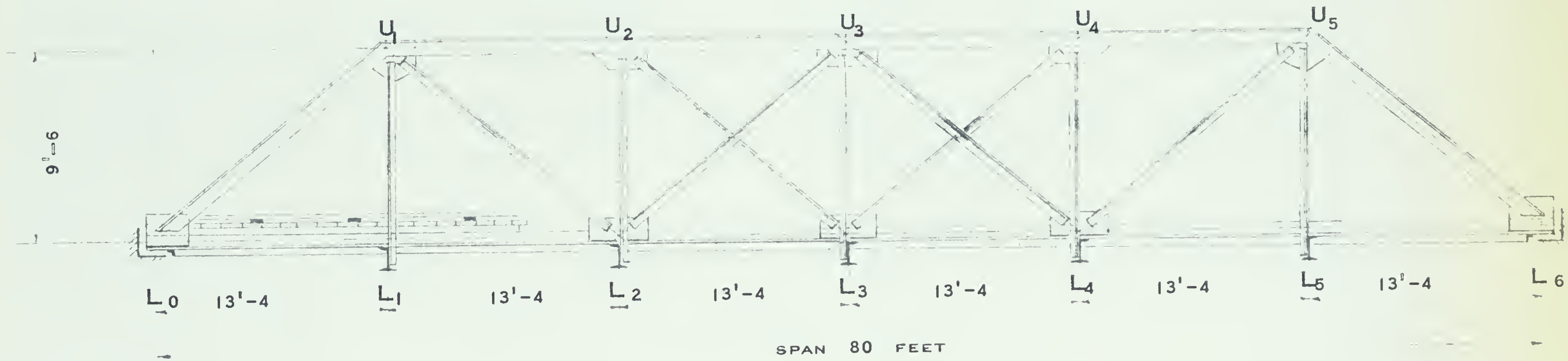

OUT
IN
20 10 0 10 20
IN
OUT
20 10 0 10 20
U5
U4
U3
U2
U1
NORTH TRUSS
SOUTH TRUSS
20 10 0 10 20
DISPLACEMENT IN INCHES
20 10 0 10 20
DISPLACEMENT IN INCHES
INITIAL LATERAL POSITIONS
OF TOP CHORDS
AT NO LOAD

Fig. 2

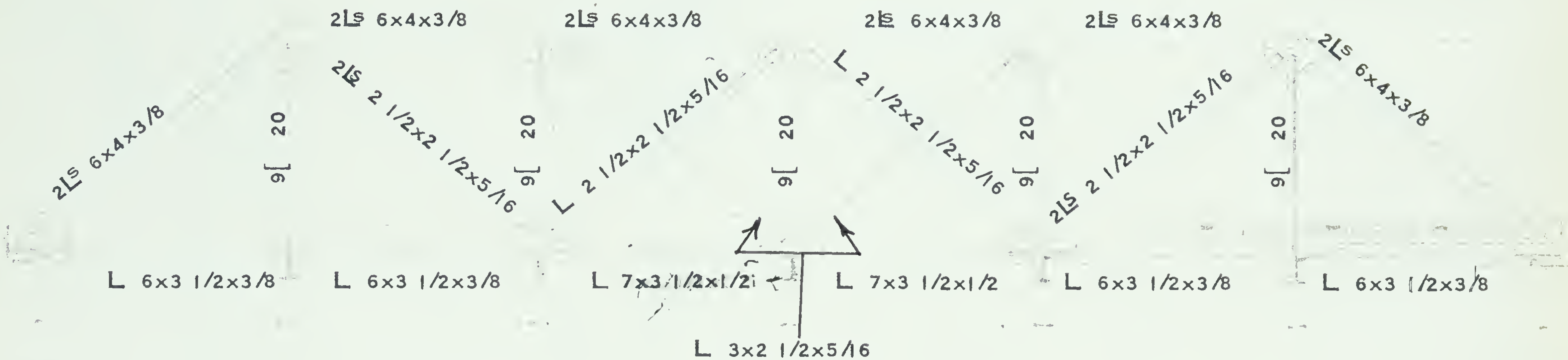


NOTE IN REFERENCE TO THE PANEL POINTS
L0 IS THE WEST END OF BOTH TRUSSES AND
L6 IS THE EAST END.

ELEVATION

SHOWING DIMENSIONS
AND
PANEL POINTS

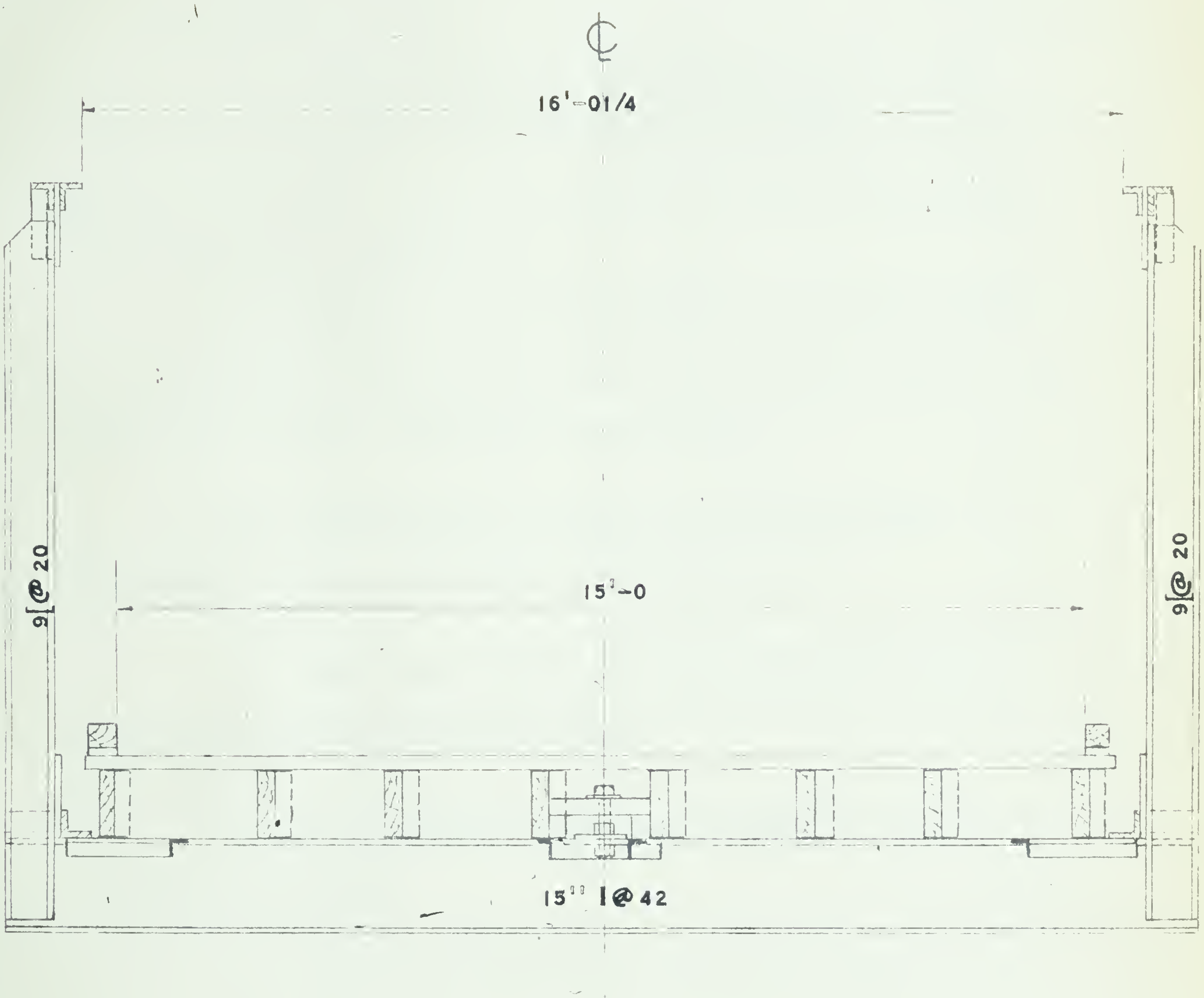
Fig. 3



ELEVATION

SHOWING THE TYPE AND SIZE OF MEMBERS

Fig. 4



SECTION
SHOWING THE TYPE AND SIZE OF MEMBERS

Fig. 5

CHAPTER II

EXPERIMENTAL PROGRAM

A. PREPARATION

I INSTRUMENTATION

The type and extent of instrumentation for a test such as this could have been quite varied, depending on the available equipment. Basically, however, it must provide data with the necessary accuracy to:

1. Determine the magnitude and type of load carried by each member of the structure. This was used as a check on the assumptions used for theoretical calculations.

2. Determine the failure load using the semi-experimental theories of Southwell and Shanley.

3. Help in the study of the characteristic behavior of such structures under applied load.

Thus the data that was deemed necessary for each increment of load was:

1. The lateral deflections of the top chord.

2. The vertical deflections of the bridge at each panel point.

3. The angle changes between the floor beam and the vertical web members, (at the knee of the "rigid" frames).

4. The axial strains of all tension members and the axial and bending strains of compression members.

(a) Lateral Deflections

Two separate methods were used to measure the lateral deflections of the top chord (see Fig. 7). One method consisted of measuring the deflections to one thousandth of an inch with dial indicators, while the other involved measuring the lateral deflections to the nearest sixteenth of an inch with a steel rule. In this way, accurate deflection readings were obtained in the low load range as well as near the failure load when the deflection , for each increment of load, was large.

There were nine of each of these measuring devices on each top chord. They were located at each panel point and midway between each panel point. These measuring devices or gauges measured the change in the distance between the top chord and a vertical plane situated to the outside of the truss, (see Fig. 6).

This vertical plane was comprised of four ten-inch wide flange steel columns driven six to eight feet into the ground. The cut-off of these columns was at approximately the same height as the top of the truss. Two lengths of 18 inch wide flange beams were placed on top of the columns with flanges

in the vertical positions. The webs of these members were tack-welded to the columns to provide stability.

The dials were supported by adjustable 1 1/2" x 1 1/2" angle brackets welded on the top chord. The plunger of the dial rested against a vertical plate which was welded to the flange of the 18 inch wide flange.

In the second method of measuring the lateral deflections, a steel rule was butted against the vertical leg of the double angle on the top chord, and the readings were taken at a plumbed wire which hung from a short bracket welded to the 18 inch wide flange.

(b) Vertical Deflections

Vertical deflections were measured at the ends and at the centre of each floor beam by means of plumbed, graduated rules affixed to the lower flange of the floor beam. The horizontal datum consisted of a constant-tension wire (see Fig. 8).

(c) Angle Changes Between The Floor Beam And The Verticals

The angle changes between the floor beams and the vertical members were obtained by means of dial gauges. Each gauge, attached to the top of the floor beam by a short post measured the horizontal movement of the vertical at a fixed distance above the top of the floor beam (see Fig. 9).

(d) Strain Measurements

"SR-4" electrical resistance strain gauges (type A-3) were used to determine the axial strains in all the members and to

indicate any bending in the compression members.

A total of 372 of these gauges were used. The number of gauges required for the various members were:

1. Single-angle member - 4 gauges (See preliminary Test #1, Appendix III):

2. Double-angle member - 8 gauges:

3. Vertical channel member - 6 gauges:

These groups of gauges were located as shown on Figs. 10, 11 and 12.

Ten gauges were affixed at each of four sections on the centre floor beam, and at each of two sections on the other floor beams. (see Fig. 13). It was reasoned that the latter would be subjected only to a constant bending moment introduced by the rotation of the verticals.

2 LOADING APPARATUS

The fixed loading structure consisted of four 10 inch wide flange steel piles driven approximately 28 feet into the ground. These piles were located symmetrically, two on each side of the bridge. (see Fig. 16). A 36 inch wide-flange beam was placed on top of the two piles on each side as a pile cap and the tops of the piles were welded all around to the flange of the pile cap. Spanning between the two pile caps was a composite beam against which the jacking load was applied.

This loading beam consisted of four 18 inch wide flange beams placed parallel to and above the centre floor beam. The bottoms of the four beams were connected by two 10 inch wide-flange members placed symmetrically six feet apart, to distribute the load of the jacks between the four beams (see Figs. 14 and 15). Thus two equal, point loads were applied to the centre floor beam. This arrangement produced a single point load at the middle panel point L_3 of the truss; on the bridge it simulated the load produced by a rear axle of a truck.

Each of the hydraulic jacks was of fifty ton capacity, and was fitted with a "Marsh" pressure gauge. These gauges were calibrated by applying a load on the jacks with a Baldwin Testing Machine. The calibration curves are shown on Figures 53 and 54.

B. TEST PROCEDURE

Approximately two weeks of preliminary testing was required to correct malfunctions in the instrumentations. The greatest source of difficulty was with the strain gauges, because differential thermal effects were produced on gauges exposed to direct sunlight as opposed to shaded gauges. To eliminate this problem, the final testing was done at night.

The testing procedure consisted of applying the load in increments and recording the deflections, angle changes and

strains for each increment of load. The loads at which readings were recorded on the day of the test to failure were: 0, 10, 20, 25, 30, 35 and 40 tons.

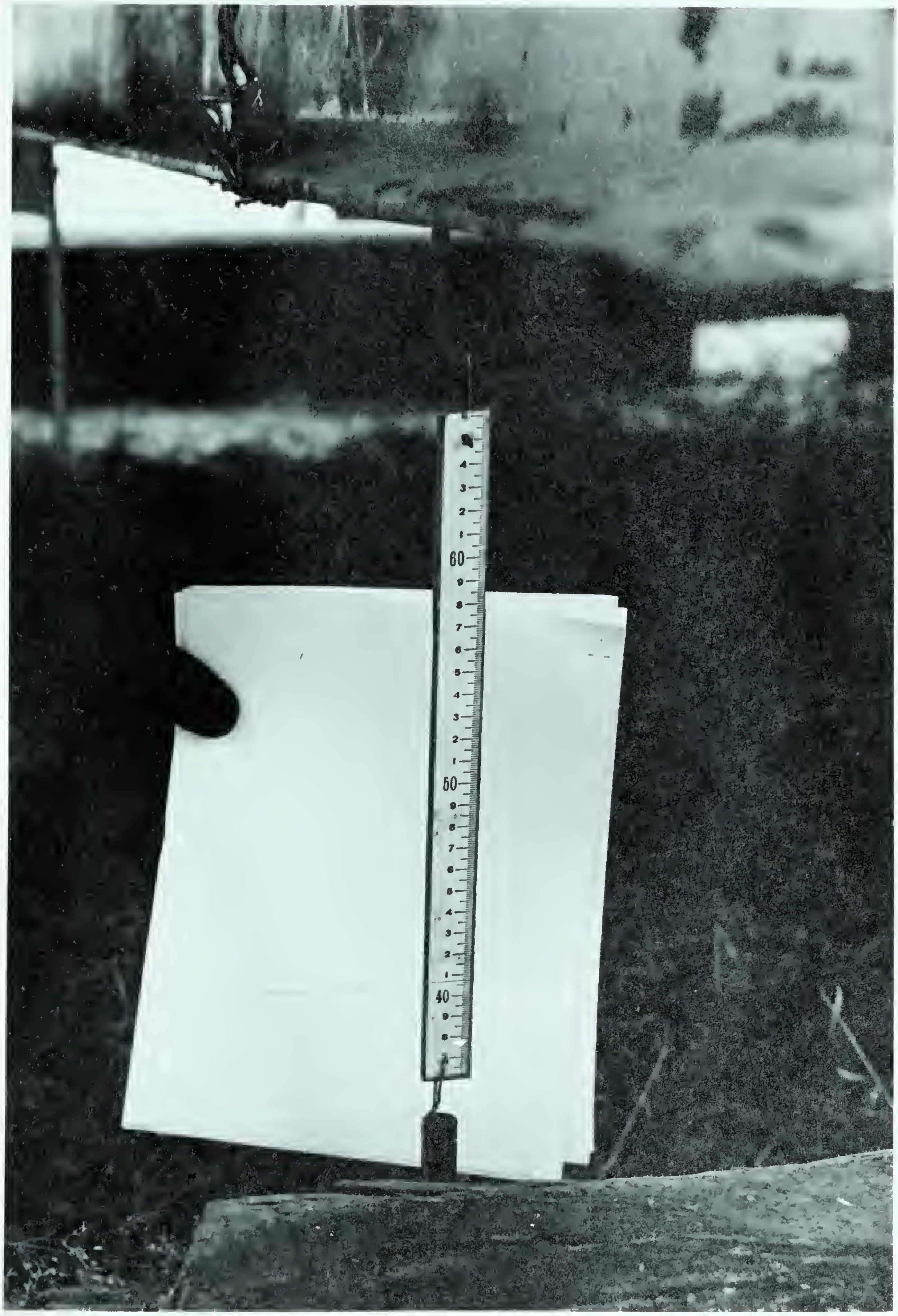
The load was applied by two operators, one for each jack. To obtain simultaneous rate of loading, the operators read the pressure gauges aloud during the jacking operation.

Photograph showing the bridge and the vertical plane which was used as a datum for measuring lateral deflections.

Fig. 6

Photograph showing the instrumentation used for measuring lateral deflections.

Fig. 7

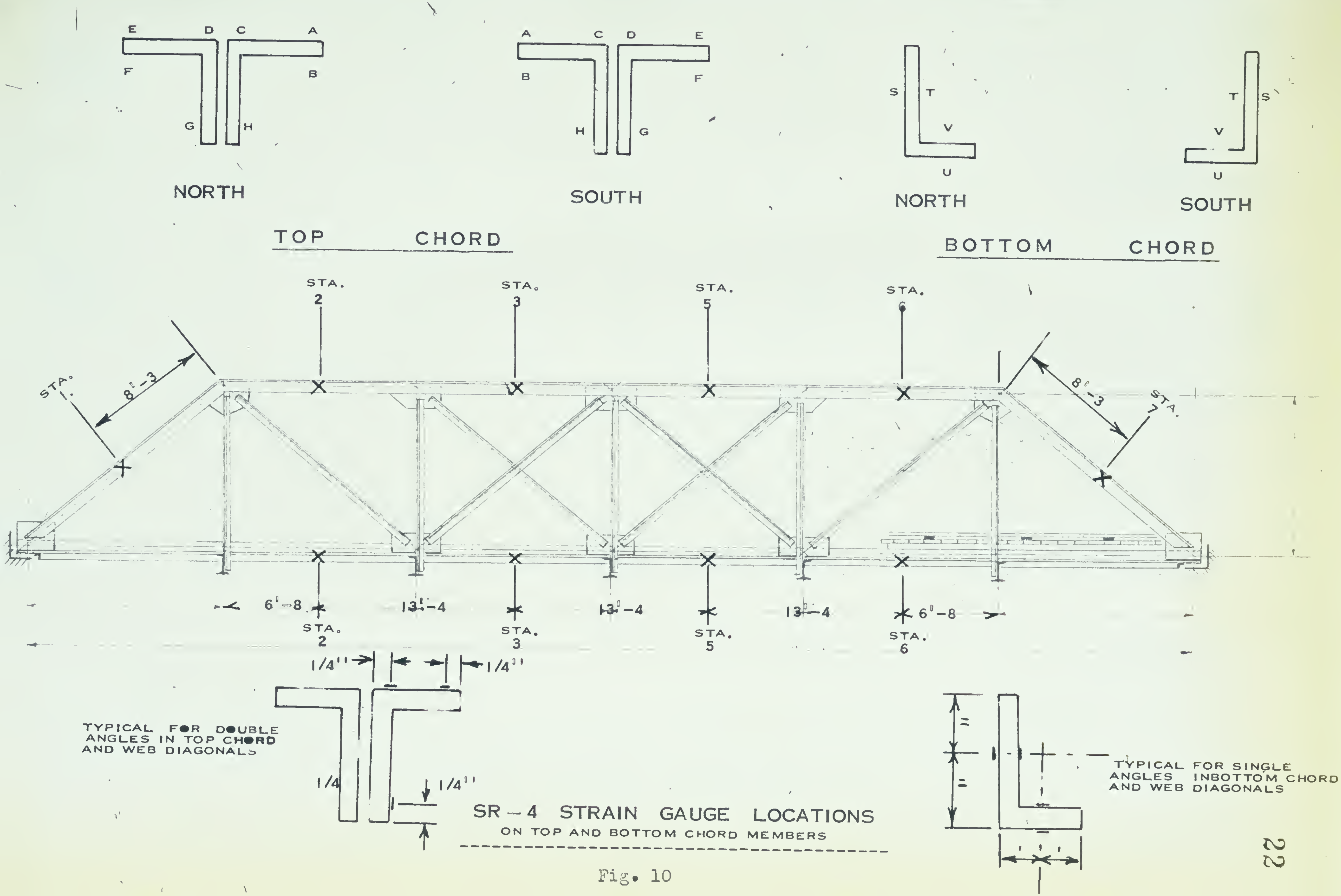


Photograph showing the instrumentation used for measuring vertical deflections.

Fig. 8

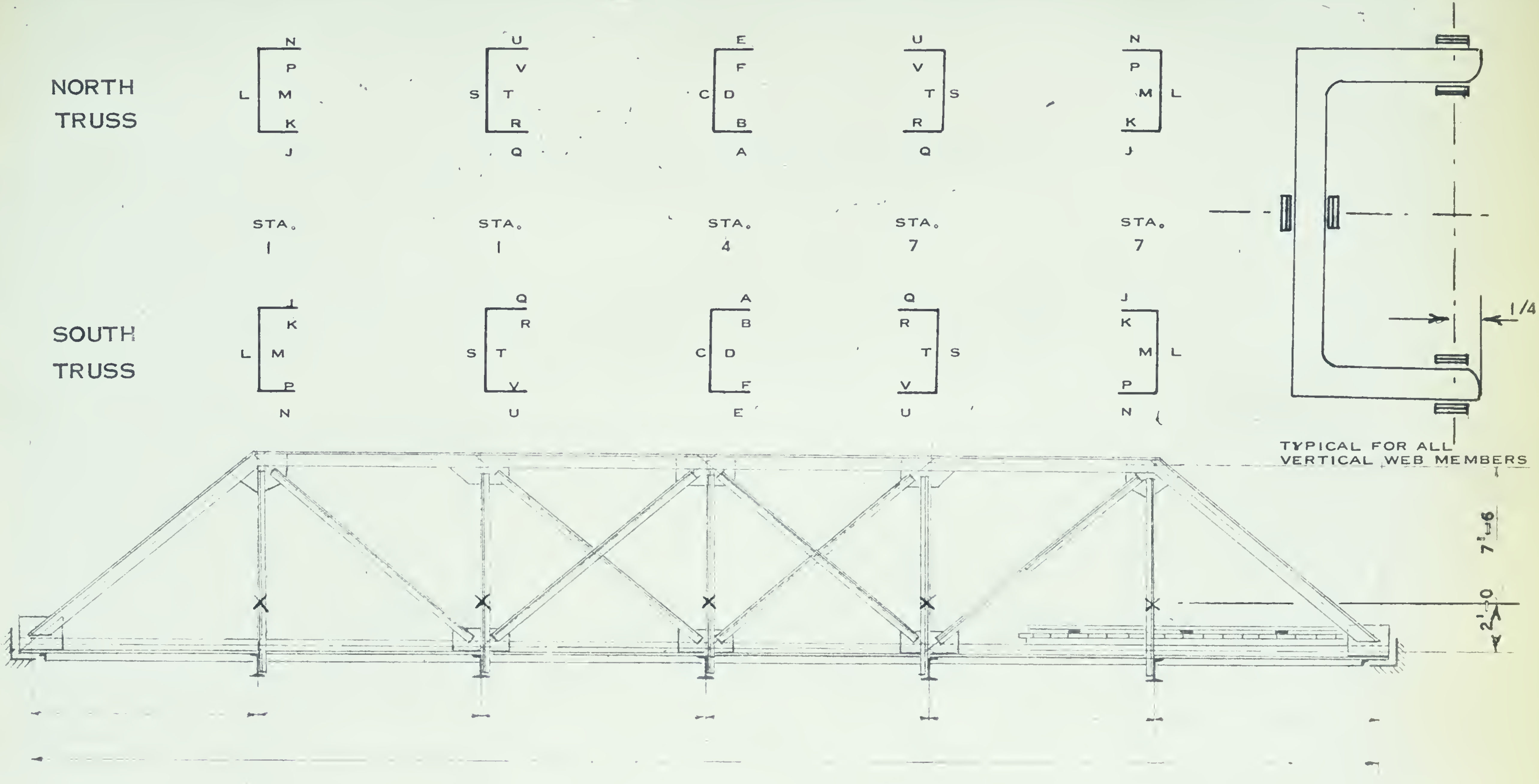
Photograph showing the instrumentation used for measuring angle changes between the vertical and the floor beam.

Fig. 9



SR – 4 STRAIN GAUGE LOCATIONS
ON TOP AND BOTTOM CHORD MEMBERS

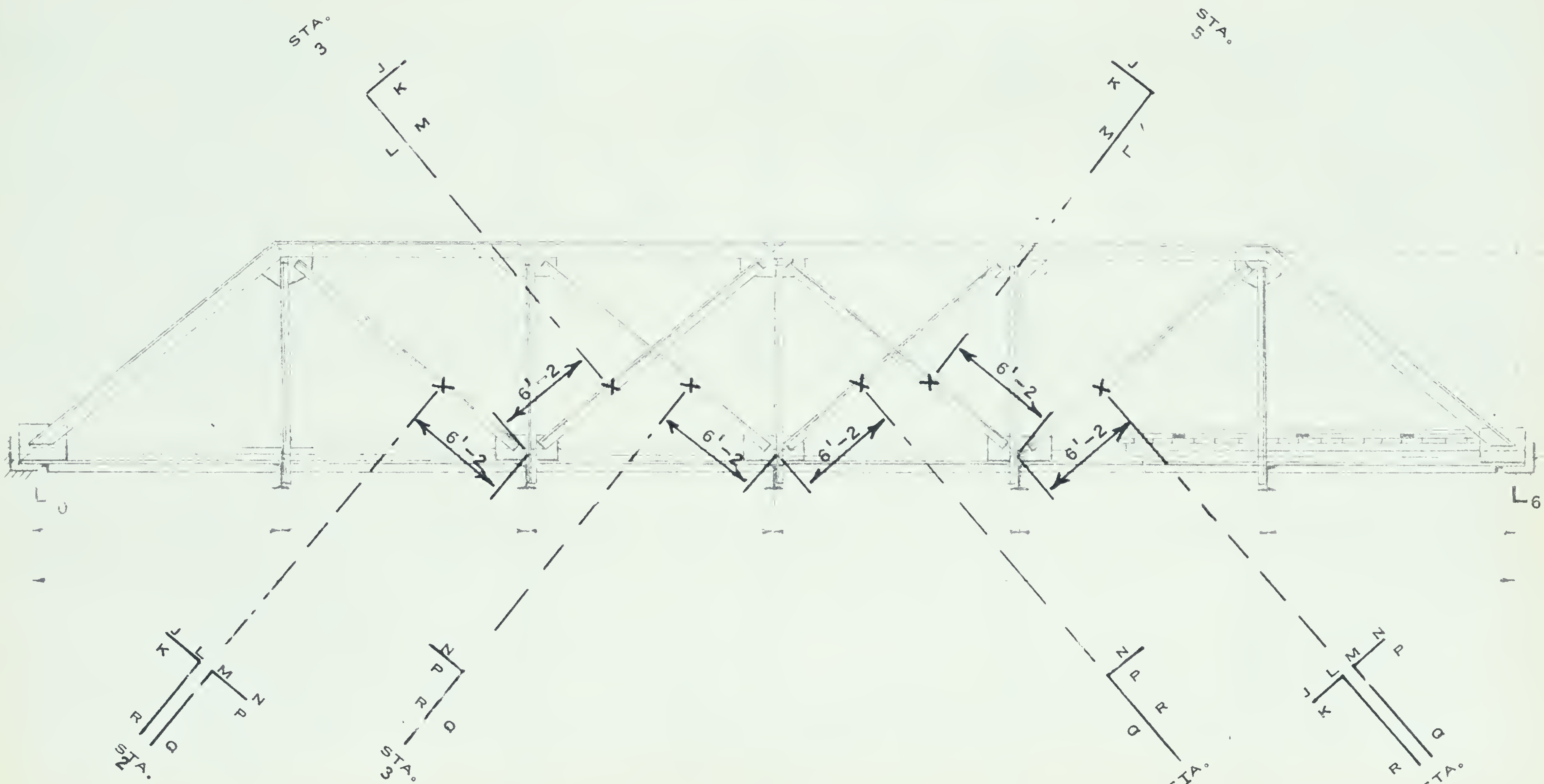
Fig. 10



SR – 4 STRAIN GAUGE LOCATIONS
ON VERTICAL WEB MEMBERS

Fig. 11

SEE TYPICAL SINGLE AND DOUBLE ANGLE GAUGE LOCATIONS
ON FIGURE ______

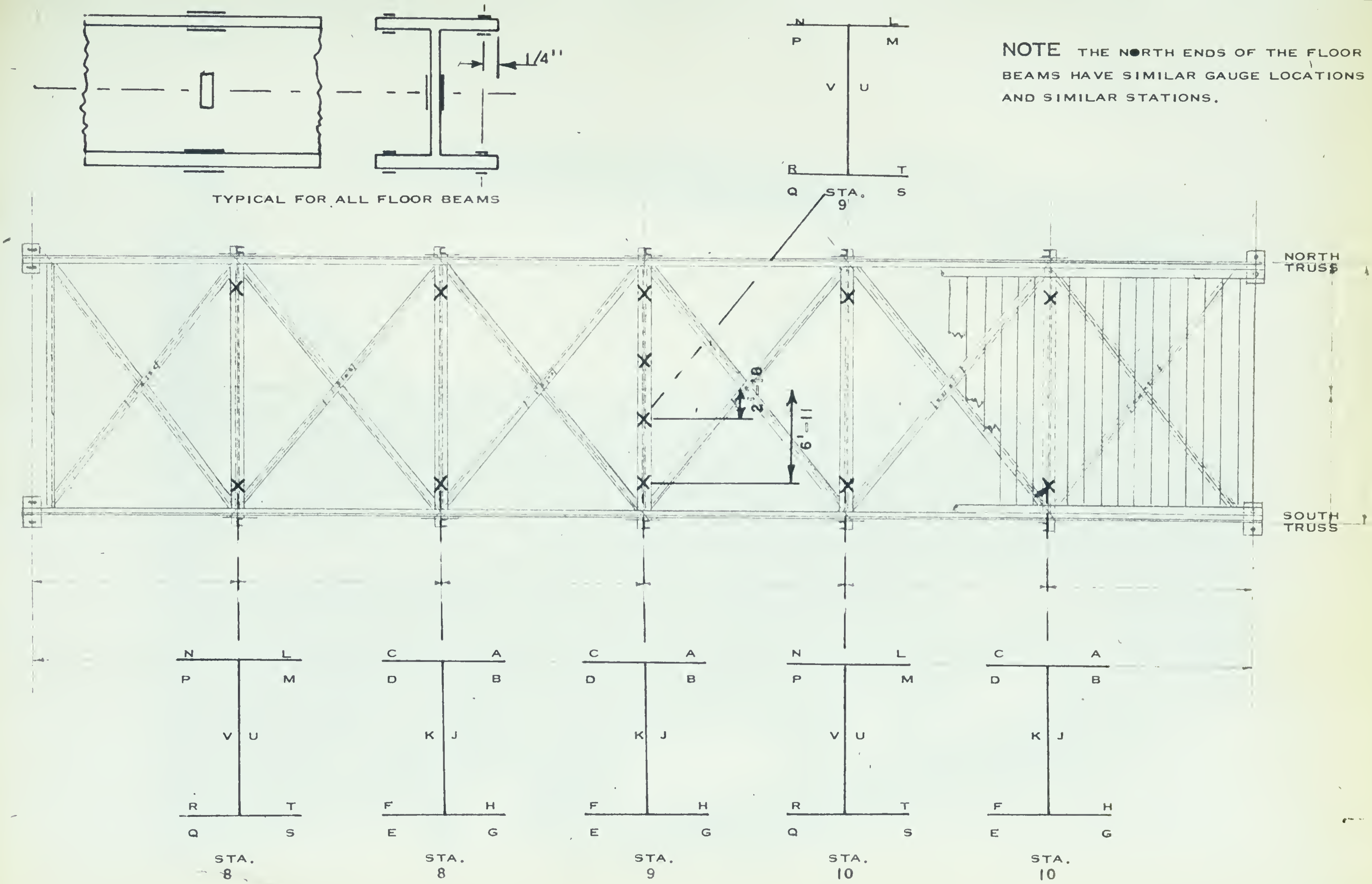


SR – 4 STRAIN GAUGE LOCATIONS

ON DIAGONAL WEB MEMBERS

Fig. 12

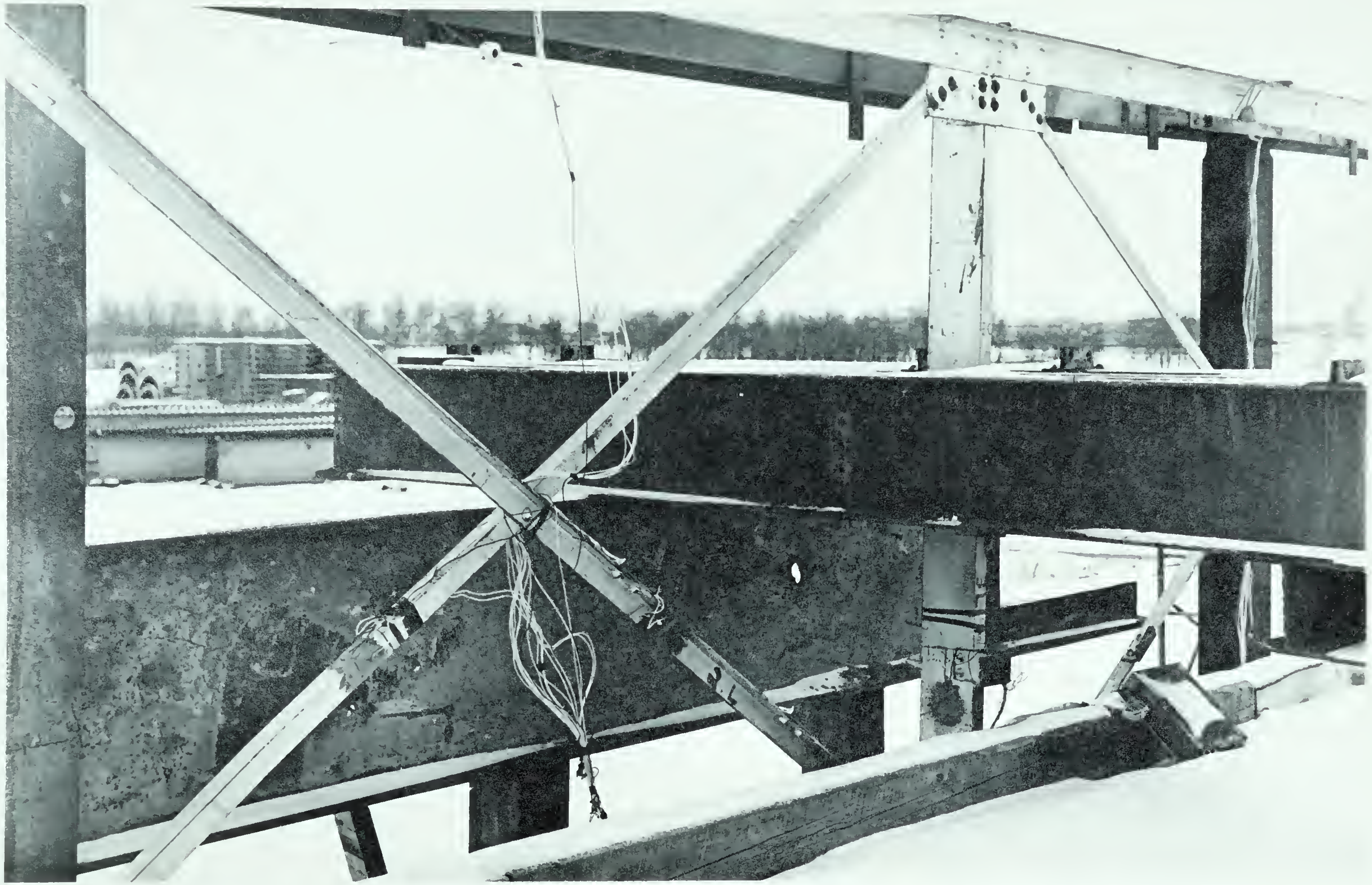
FIG. 12



SR – 4 STRAIN GAUGE LOCATIONS

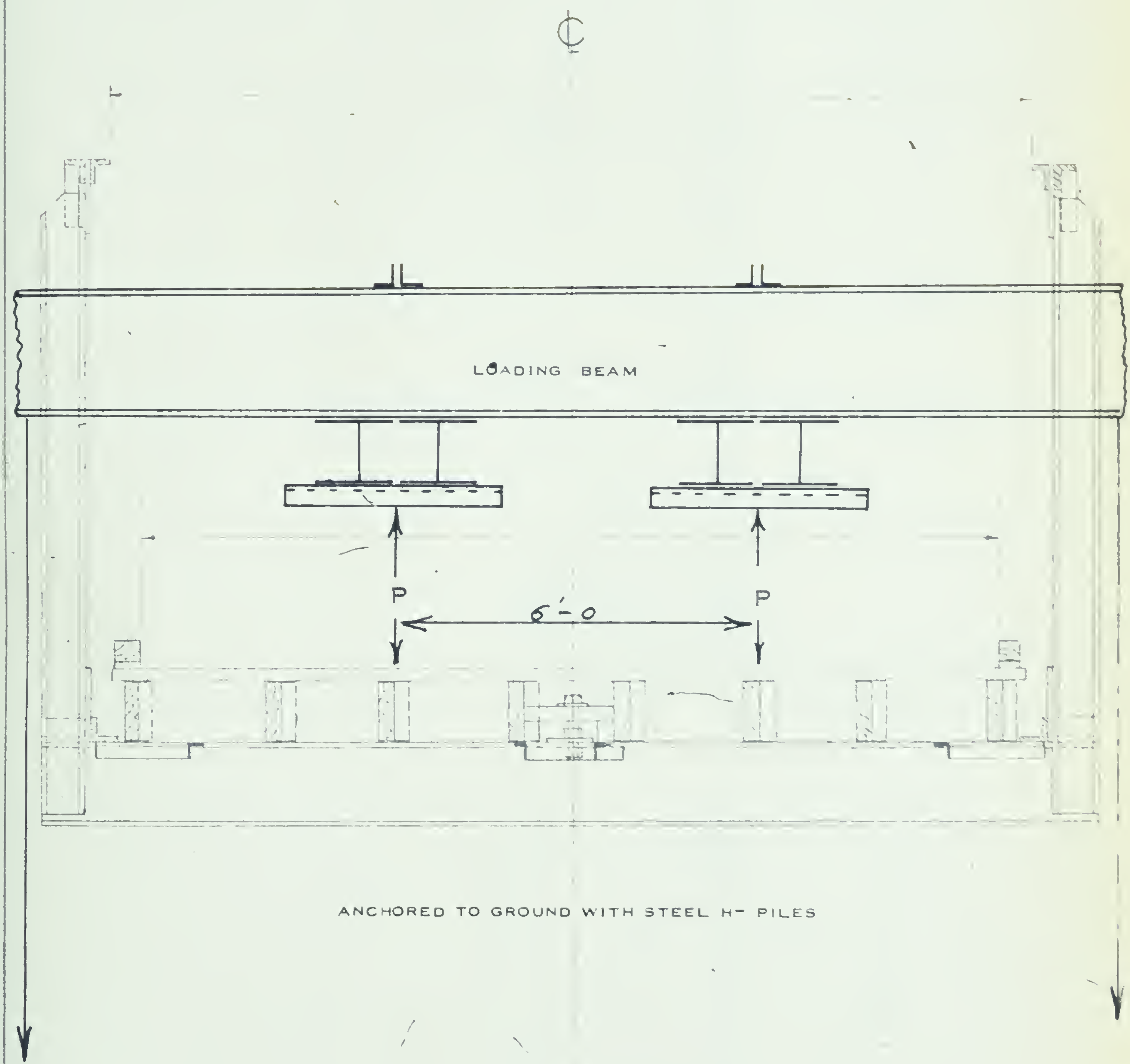
ON THE FLOOR BEAMS

Fig. 13



Photograph showing the loading beam and the pile-cap, as seen from the bridge.

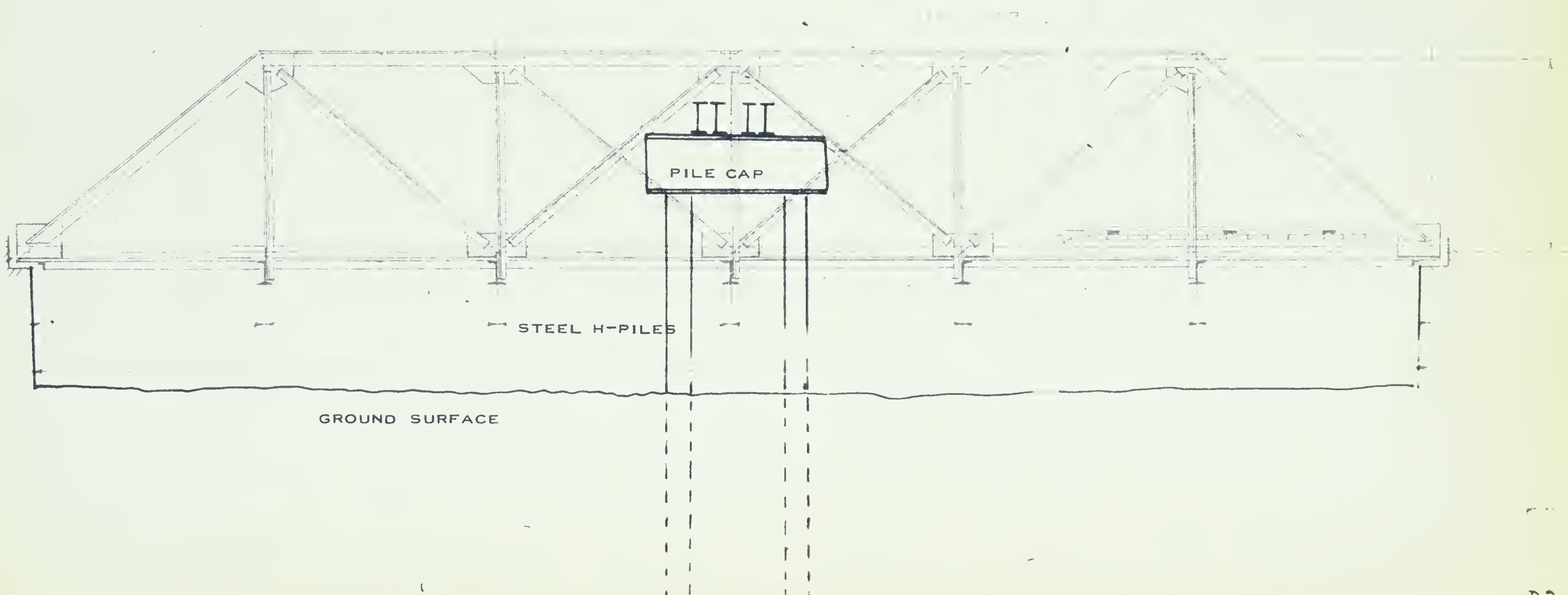
Fig. 14



LOADING APPARATUS

END VIEW

Fig. 15



LOADING APPARATUS
SIDE VIEW

CHAPTER III

RESULTS

A. PRESENTATION OF RESULTS.

I ULTIMATE LOAD

The failure load was realized by the operators of the jacks, when the load indicator dials failed to register increase in load even though jacking continued. On each of the jacks the maximum load attained was 21 tons. The jacking operation continued for some time after the indicator needle stopped. Ultimately jacking was discontinued because there was no increase in load and also because the south jack had run out of travel.

At the same time as the jacks indicated that the ultimate had been reached, observers watching the top chord were also convinced that failure load was attained. By visual observation, the lateral deflections during this increment of loading were larger than any previous increment of loading. The lateral deflections at and adjacent to the centre panel point consisted of the complete travel of the dial indicators plus a substantial gap which was visible between the end of the dial plunger and the plate against which the plunger should bear.

The observed failure load was 42 tons. However, no strain or deflection data was obtained at this load because ten

to fifteen minutes after this load was attained, a plastic hinge developed on the top chord of the south truss between panel points U_2 and U_1 (in the region where the large initial deflections existed). This yielding of the south top chord caused such large lateral deflections of the top chord, that for practical purposes complete failure had occurred. The load at which the bridge finally came to equilibrium was only 17 1/2 tons. This load was distributed as 13 1/2 tons on the north jack and 4 tons on the south jack.

From the observed failure, it can be stated that both trusses did fail, even though no deflection data was obtained at the failure load. The plots of load versus lateral deflection of the top chords (Fig. 17) indicate that both trusses are approaching failure. However, it cannot be said that both trusses failed simultaneously. It is possible that the south truss failed first, causing a sufficient shift of load onto the north truss to produce a subsequent failure of the latter. This action was not detected by the operators of the jacks.

The failure loads obtained with the aid of available theories, and experimental data where necessary, are presented in the following table along with the test results. These results are expressed both as the live applied load on truss and as the live plus dead axial load on the central portion of the top chord.

Sample calculations for each of the theories may be found in Appendix I.

	LIVE LOAD ON TRUSS AT L_3	AXIAL LOAD ON U_2-U_4 (live plus dead)
Observed test results	42 kips	109.6 kips
Southwell's theory		
North truss	49.8 kips	123.5 kips
South truss	42.6 kips	111.0 kips
Timoshenko's theory	62 kips	145.0 kips

The results obtained with the aid of Southwell's theory are within 2 percent of the experimental results for the south truss. The failure load prediction for the north truss is 8 percent higher. This could mean that there was a shift of load from the south truss to the north truss and thus a premature failure of the latter occurred due to the extra load.

The results obtained with the aid of Timoshenko's strain energy equation can also be said to be reasonable, though they are not as close to the test results as that obtained with Southwell's theory. This critical load could be greatly reduced if it were possible to consider the eccentric loading as it existed prior to the buckling of the top chord. (See the deflections of the top chord for the forty ton load interval on figure 20). However, it was necessary to assume that the top chord was

perfectly straight prior to buckling, in order to make the theoretical calculations.

2. GENERAL BEHAVIOR OF THE STRUCTURE

The structure is constructed in such a manner that no actual load can be applied in the planes of the trusses. That is, the load is applied directly onto the horizontal member of a rigid frame which consists of the floor beam and the two verticals. Thus the actual load applied on the truss consisted of a load in the plane of the truss and a moment introduced by the fixity of the floor beam connection to the truss.

To understand the effect of this type of loading, it is necessary to consider only this rigid frame. Ordinarily, the purpose of the frame, among other things, is to hold the truss in a vertical position. However, when a load is applied on the floor beam at some distance from the vertical plane of the truss, the beam will deflect under this load. This beam deflection results in a rotation of the rigid connection and consequently a lateral deflection of the top of the vertical web member (see Figs. 18 and 19). This lateral movement of the top of the vertical has the following effects.

1. It produces eccentric column loads, because elastic, lateral deflections are produced by the beam action.

2. During the buckling of the top chord the lateral restraint produced by the elastic support is decreased because the effective deflection of the elastic support is decreased. It is assumed that the direction of deflection of the top of the web vertical is the same as the deflection of the top chord, since the elastic deformations start with the first increment of load.

This behavior can be better realized by examining some of the results from the experimental data. The plots of Flexural strain on the web vertical vs. the Deflection of the top of the vertical, Figs. 45 to 49, help in understanding the behavior, and were plotted to determine the spring constants of the lateral supports. These spring constants were determined in the following manner.

1. From the "SR-4" strain gauge data the lateral bending moment at the gauge section was determined.

2. From the lateral deflection data, the deflection of the vertical for the load corresponding to the above mentioned bending moment was obtained.

Thus the lateral load on the vertical can be calculated from the known bending moment. (Assuming the elastic support is a cantilever beam with a point load on the end). The spring constant is this load divided by the measured lateral deflection.

To obtain an average value of the spring constant applicable over the full loading range, the following procedure was used:

1. Plot the fiber strains in micro-inches per inch, due to lateral bending versus the lateral deflections for each load increment.

2. Obtain the slope of the best fitted straight line. The units of the slope constant are micro-inches/inches^2.

3. The spring constant is obtained by multiplying the slope by the expression $\frac{ES}{L}$. Thus the spring constant $C = \frac{ES}{L} \times \text{slope} \times 10^{-6}$.

Where: $E = 30 \times 10^6$ psi.

$S = 13.5$ in^3 (Section Modulus)

$L = (7.5 \times 12)$ inches (the distance from the top of the vertical to the locations of the strain gauges).

These plots are shown on Figs. 45 to 49 and the spring constants were determined to be 1590 pounds per inch for L_4-U_5 (the verticals adjacent to the centre panel point), and 2050 pounds per inch for L_5-U_5 (the end verticals). This compares with 2160 pounds per inch, the theoretical spring constant. (See sample calculations, Appendix I). The reason for the higher spring constants for the end verticals is that the stiffness of the floor beam is increased by the dead load of the decking, since the floor beam deflects upwards during the buckling of the top chord. It is necessary to point out that the theoretical spring

constant was determined on the assumption that the joint between the vertical and the floor beam is rigid. Since this is not so, it is not surprising that the true spring constant of the lateral support adjacent to the centre panel point is somewhat less than 2160 pounds per inch.

Due to the initial deflections in the south truss, the behavior of the rigid frames at panel points 1 and 2 was very irregular. As a result, no spring constant could be determined for these lateral supports. Also the spring constants cannot be obtained for the centre verticals, because the effective deflection is decreased. The best that can be done with the centre verticals is to assume the spring constant is zero since the bending is reversed and becomes zero near the failure load (See Fig. 47).

Figures 45 to 49 also confirm the assumption that the centre floor beam carries all the load, that is, the decking does not distribute load to other floor beams. To explain this conclusion it must be noted that the centre floor beam yielded in the 25 to 30 ton load interval. If distribution were present, the beams adjacent to the centre one would carry a higher percentage of the applied load increment after yielding of the centre beam than before. This should cause a change in the spring constant. Such a change cannot be detected in the above noted plots.

As a result of this observed behavior, and the information obtained from the experimental data, it was assumed in the application of Timoshenko's theory that the centre vertical member supplied no restraint to buckling of the top chord. However, the beam action that existed on the top chord prior to buckling could not be considered in the calculations, and also the magnitude of the lateral support, used with Timoshenko's theory, is somewhat high because the joint at the knee was considered as rigid. Both of the last mentioned factors will cause the theoretical failure load to be higher than the actual failure load.

The measurement of angle changes between the floor beam and the vertical member also give significant results. No direct use was made of these measurements, but they have been used as a check on the performance of the bridge under load.

A plot showing these angle changes is shown in Figs. 18 and 19, along with a plot showing the relative bending of the same vertical member. The bending of the vertical is obtained from the strain gauge data.

A better understanding of the behavior of the bridge is possible from this information. These plots clearly show how the centre vertical is forcing the top chord in, while the adjacent verticals are restraining the top chord.

B. DISCUSSION OF RESULTS

In general the results obtained from the instrumentation used may be said to be satisfactory. They give a good indication of the behavior of the structure under this particular loading. Of major significance are the deflections of the top chord and the strains obtained from the "SR-4" Gauges.

(a) Lateral Deflection Measurements

The modes of deflection of the top chord are shown on Fig. 20. Due to the initial deflections of the south chord, the deflections on this plot of the south truss are not as symmetrical as those of the north truss.

(b) Strain Measurements

A comparison of the measured axial strain versus the theoretical strain is shown on Figs. 37 to 44, as a plot of the load on the bridge versus the axial strain in each member. Since the bridge is symmetrical and since it was symmetrically loaded, there are four similar members which would be expected to have the same strain for each value of load. The strains in the four similar members along with the theoretical strain are shown on the graphs. In most cases the actual strains are within a few percent with the theoretical values. The only exceptions are members L_2-U_3 on both the north and south sides,

and U_3-L_4 on the north side. These three members were initially deformed. On the other hand U_3-L_4 on the south truss was initially straight, and the actual strains for this member are within 2 percent of the theoretical values.

It can be stated that the only available theory with which an exclusively mathematical failure load prediction can be made is the strain energy equation. However, this theory has its limitations, particularly when the loads are not applied in the plane of the truss. Because of these limitations and because the loading used in the test was not applied in the plane of the truss, test data had to be used to obtain the critical load.

The observations bear out the statement made by Bleich that the mode of buckling and the failure load may be affected by different types of loads. He stated that a uniform load across the bridge would produce behavior different from that obtained when only one or several floor beams carry the load. It is easily seen that a uniform load would produce much the same condition as loading in the plane of the truss.

The statement made by Hrenikoff that the mode of buckling would not be greatly affected by different loadings is not verified in this test nor by the theoretical calculations. It has been observed in this test that different loading may greatly

affect the restraining forces produced by the lateral supports, and the theoretical calculations indicate that these changes in the lateral supports alter the mode of buckling and the failure load. However, it may be possible that different loadings in the plane of the truss will not greatly affect the mode of buckling, and perhaps Hrenikoff is considering only loadings in the plane of the truss.

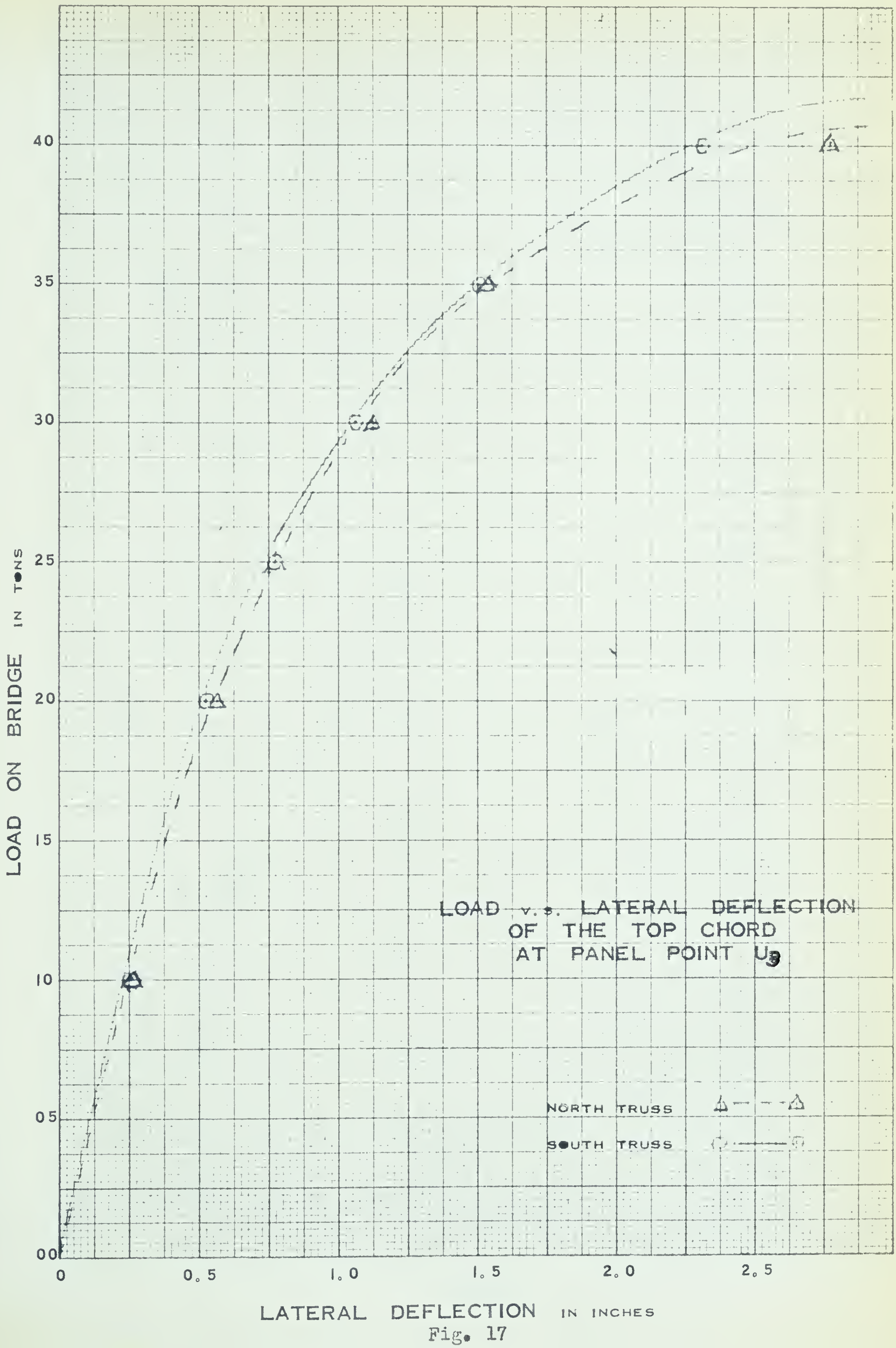
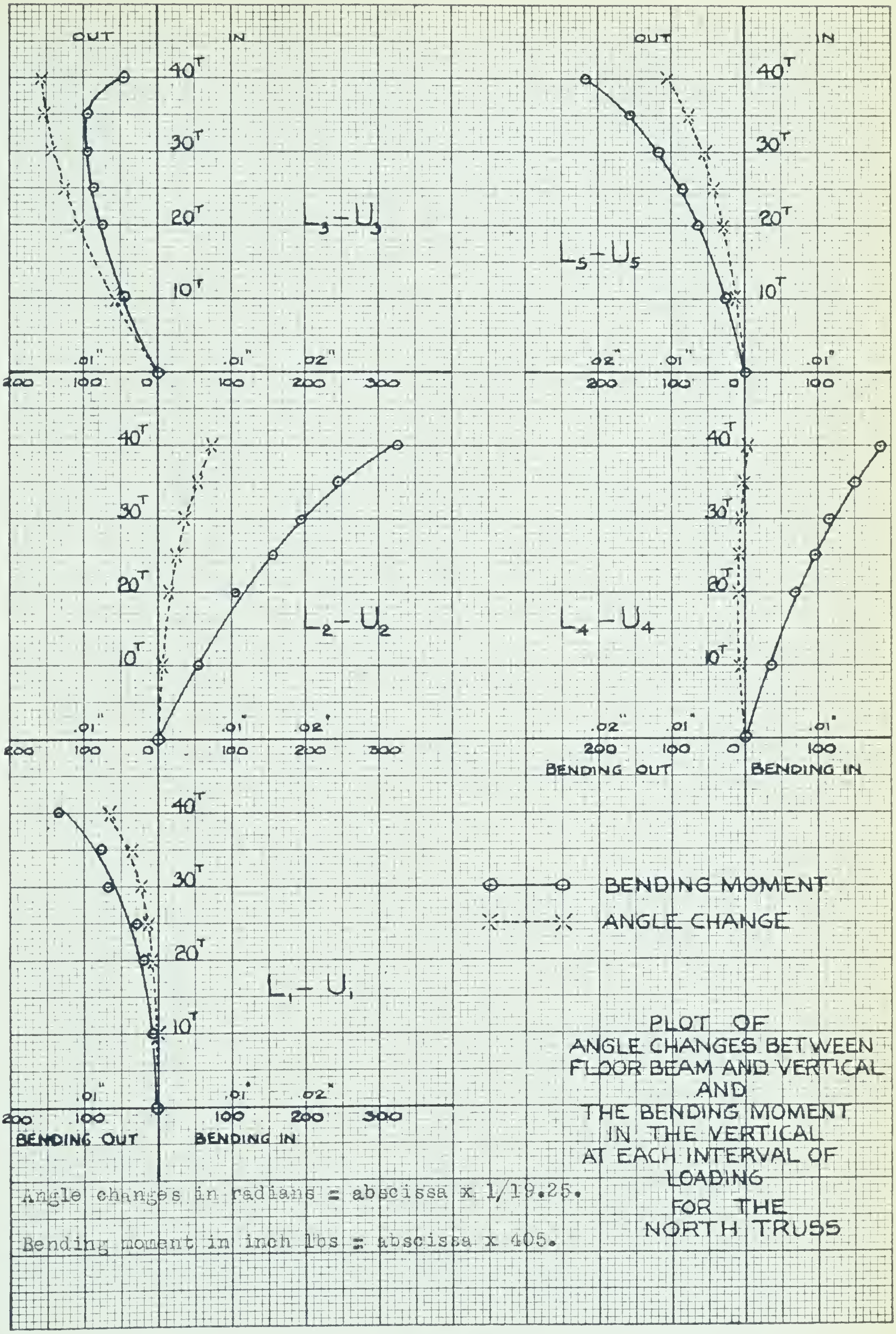
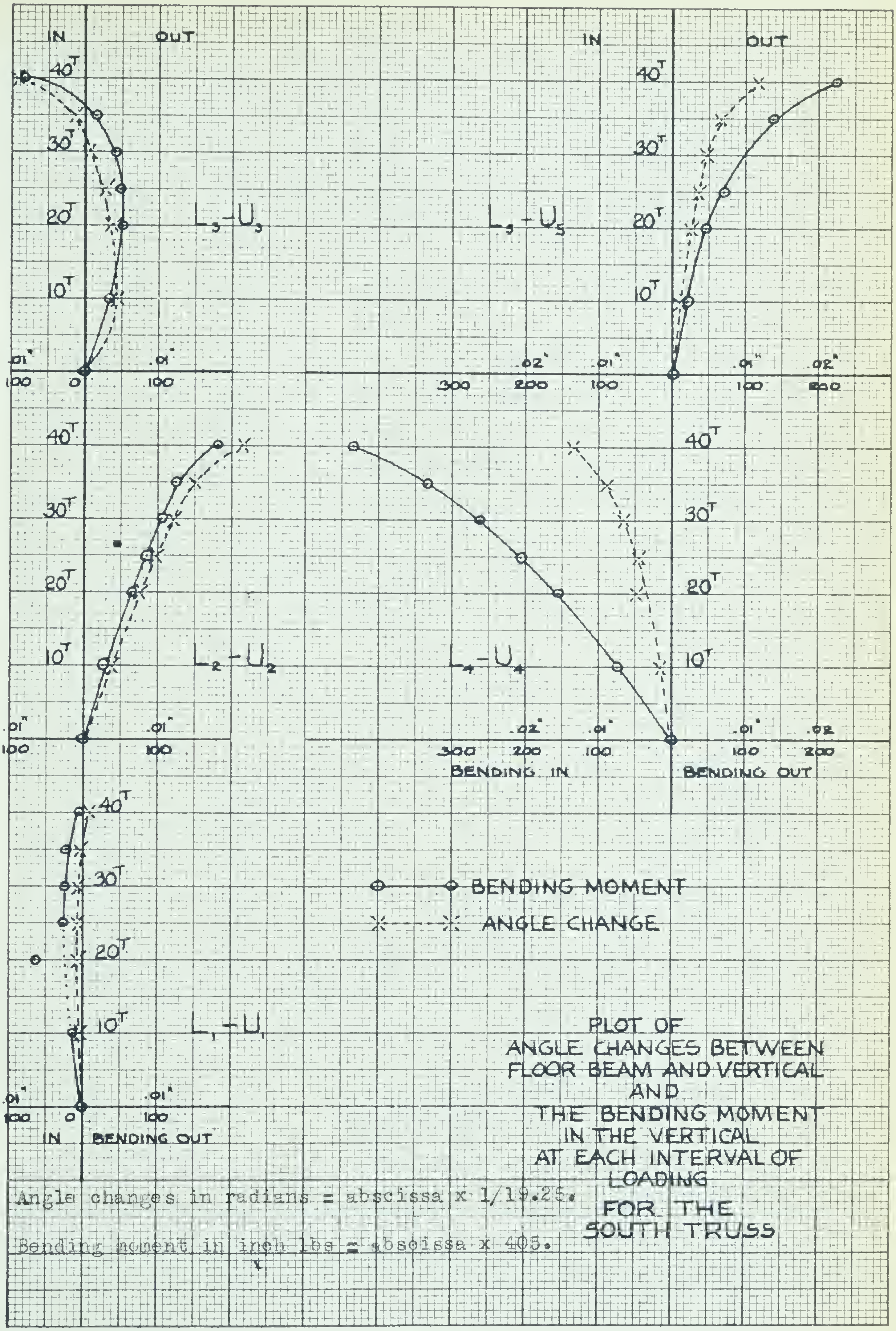

LOAD ON BRIDGE IN TONS
40
35
30
25
20
15
10
05
00
0
0.5
1.0
1.5
2.0
2.5
LOAD v.s. LATERAL DEFLECTION
OF THE TOP CHORD
AT PANEL POINT U3
NORTH TRUSS
SOUTH TRUSS
LATERAL DEFLECTION IN INCHES

Fig. 17



OUT
IN
$L_3 - U_3$
$L_5 - U_5$
$L_2 - U_2$
$L_4 - U_4$
$L_1 - U_1$
BENDING OUT
BENDING IN
BENDING MOMENT
ANGLE CHANGE
PLOT OF
ANGLE CHANGES BETWEEN
FLOOR BEAM AND VERTICAL
AND
THE BENDING MOMENT
IN THE VERTICAL
AT EACH INTERVAL OF
LOADING
FOR THE
NORTH TRUSS
Angle changes in radians = abscissa x 1/19.25.
Bending moment in inch lbs = abscissa x 405.

Fig. 18



IN
OUT
L_3-U_3
L_5-U_5
L_2-U_2
L_4-U_4
L_1-U_1
BENDING IN
BENDING OUT
BENDING MOMENT
ANGLE CHANGE
PLOT OF
ANGLE CHANGES BETWEEN
FLOOR BEAM AND VERTICAL
AND
THE BENDING MOMENT
IN THE VERTICAL
AT EACH INTERVAL OF
LOADING
FOR THE
SOUTH TRUSS
Angle changes in radians = abscissa x 1/19.25.
Bending moment in inch lbs = absoissa x 405.

Fig. 19

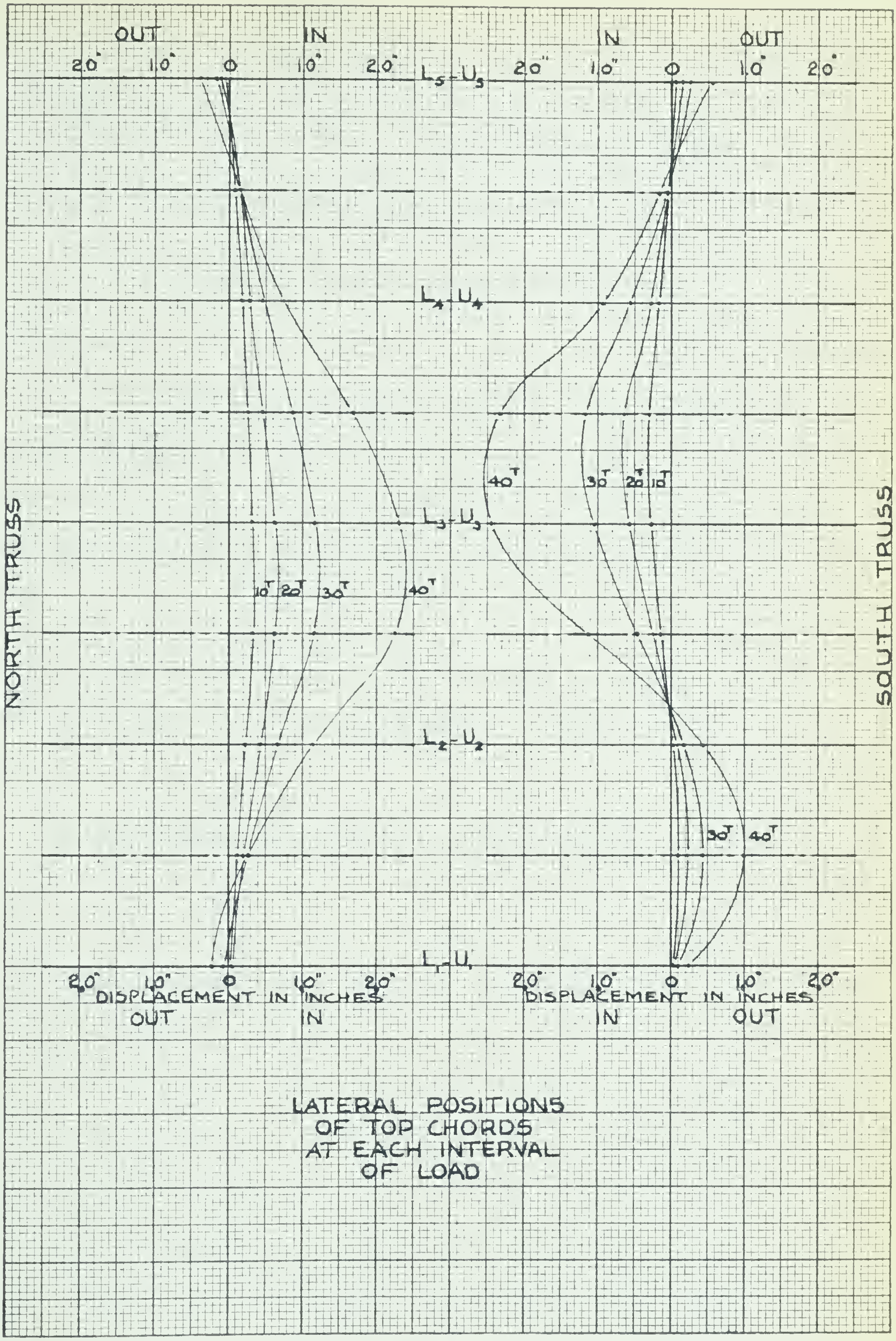

OUT
IN
IN
OUT
2.0" 1.0" 0 1.0" 2.0"
L_5-U_5
2.0" 1.0" 0 1.0" 2.0"
L_4-U_4
L_3-U_3
L_2-U_2
L_1-U_1
NORTH TRUSS
SOUTH TRUSS
10T 20T 30T 40T
40T 30T 20T 10T
30T 40T
DISPLACEMENT IN INCHES
OUT IN
DISPLACEMENT IN INCHES
IN OUT
LATERAL POSITIONS
OF TOP CHORDS
AT EACH INTERVAL
OF LOAD

Fig. 20

CHAPTER IV

CONCLUSIONS

The results of this test may be summarized as follows:

1. The failure load of the bridge was 42 tons.

2. The failure is a result of the buckling of the top chord.

3. The strain energy equation could be a satisfactory theory for predicting the capacity of pony-truss bridges, if the deformed shape can be determined, and if the rigidity of the joint between the vertical and the floor beam can be correctly assumed.

4. The initial deformations in the used bridge seem to affect the behavior of the structure and its ultimate load capacity.

Further testing is necessary to obtain more conclusive results.

CHAPTER V

RECOMMENDATIONS FOR FUTURE TESTS

Many tests could be carried out very economically on a structure composed of straight members, since failures due to elastic instabilities do not appear to produce any permanent deformations on the structure after the load is removed. As has occurred in this test, the north chord was loaded to failure and then returned to its normal position when the load was removed. Therefore it appears possible to load test this type of structure to failure without actually producing any permanent deformations on the truss. Blocking should be placed under the floor beams to limit the deformations due to buckling when the structure fails.

A. TYPE OF TESTS.

Several different loadings may be applied to the bridge with the existing loading beam. With slight modifications to the loading beam, the load may be applied on two or three floor beams, in much the same manner as was carried on one floor beam (centre floor beam) in this test. This loading may so be arranged that each floor beam will carry equal portions of the load or if desired, varying portions of load. The application of the load on only one floor beam should not be considered because the floor beam may yield in bending before the trusses buckle. Yielding of the floor beam occurred in this test at a load of

25-30 tons. The loads can also be applied directly within the planes of the trusses. These too could be applied in equal or varying intensities on one, two or three panel points of the truss.

Loading in the plane of the truss could be realized in practice by modifying the present bridge structure. Since this is possible, more consideration should be given to tests with the loading in the plane of the truss, because:

1. An exclusively mathematical analysis is possible for such loading;

2. Indications are that loading in the plane of the truss will greatly increase the load capacity of the pony-truss bridge.

B. REQUIRED INSTRUMENTATION

No extra instrumentation would be necessary for any of the tests. All the instrumentation that was used in this test would be sufficient. However, if a check of the portion of load carried by each member is not required, the only SR-4 strain gauges that would be necessary are those on the top chord and on the vertical web members (see Figs. 10 and 11). The remainder of the instrumentation would all be beneficial, and the only suggestion is that the dials measuring lateral deflections of the top chord should all be of one inch travel

and measure only to a thousandth of an inch. Deflections measured to one-ten thousandth of an inch are not necessary and are definitely difficult to read.

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2. Jasinsky is credited for his contribution by S.P. Timoshenko in "HISTORY OF STRENGTH OF MATERIALS, McGraw-Hill, and by F. Bleich in BUCKLING STRENGTH OF METAL STRUCTURES", McGraw-Hill.

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APPENDIX I

GENERAL THEORY & SAMPLE CALCULATIONS

The theory of simply-connected, or pin-connected trusses has been used to obtain the influence lines for the trusses. Since the trusses are statically indeterminate to the second degree, the method of work was used to obtain the stresses of the redundant members in the two central panels. The influence lines are not shown but rather the load coefficients are obtained for every truss member for unit loads at each panel point. These load coefficients are summarized in Table I.

The nomenclature used, is tabulated below with the corresponding units, where applicable.

DIAGONAL - Sloping web member.

VERTICAL - Vertical web member.

TOP CHORD OR UPPER CHORD - Members forming the upper perimeter of the truss.

BOTTOM CHORD OR LOWER CHORD - Members forming the lower perimeter of the truss.

LOAD ON BRIDGE (live load) - The load applied on the bridge with the hydraulic jacks.

INFLUENCE COEFFICIENT - Total stress in a member due to a unit load on the bridge.

STRAIN - Unit strain, micro-inches/inch.

δ - Amplitude in inches or sine curve for a regular buckling mode.

Δ - Lateral deflection of top chord in inches.

AXIAL LOADS IN MEMBERS DUE TO UNIT LOADS AT THE PANEL POINTS

Member	Load At L_1	Load At L_2	Load At L_3	Load At L_4	Load At L_5
$L_0 - U_1$	-1.438	-1.150	-0.863	-0.575	-0.288
$L_0 - L_1$	+1.168	+0.934	+0.701	+0.467	+0.234
$U_1 - L_1$	+1.000	0.000	0.000	0.000	0.000
$L_1 - L_2$	+1.168	+0.934	+0.701	+0.467	+0.234
$U_1 - U_2$	-0.934	-1.868	-1.403	-0.934	-0.468
$U_1 - L_2$	-0.288	+1.150	+0.863	+0.575	+0.288
$U_2 - L_2$	+0.073	+0.145	-0.292	-0.186	-0.094
$U_2 - U_3$	-0.835	-1.665	-1.812	-1.194	-0.600
$L_2 - L_3$	+0.802	+1.604	+1.695	+1.141	+0.571
$L_2 - U_3$	+0.163	+0.325	-0.359	-0.254	-0.126
$U_2 - L_3$	-0.125	-0.250	+0.504	+0.321	+0.162
$U_3 - L_3$	-0.021	-0.041	+0.416	-0.041	-0.021
$U_4 - L_3$	+0.162	+0.321	+0.504	-0.250	-0.126
$L_4 - U_3$	-0.126	-0.254	-0.359	+0.325	+0.163
$L_4 - L_3$	+0.571	+1.141	+1.695	+1.604	+0.802
$U_4 - U_3$	-0.600	-1.194	-1.812	-1.665	-0.835
$U_4 - L_4$	-0.094	-0.186	-0.292	+0.145	+0.073
$U_5 - L_4$	+0.288	+0.575	+0.863	+1.150	-0.288
$U_5 - U_4$	-0.468	-0.934	-1.403	-1.868	-0.934
$L_5 - L_4$	+0.234	+0.467	+0.701	+0.934	+1.168
$U_5 - L_5$	0.000	0.000	0.000	0.000	+1.000
$L_6 - L_5$	+0.234	+0.467	+0.701	+0.934	+1.168
$L_6 - U_5$	-0.288	-0.575	-0.863	-1.150	-1.438

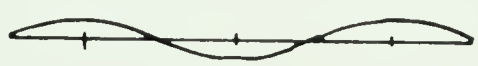
Table 1

SAMPLE CALCULATIONS (Timoshenko's Theory)

Before presenting a sample calculation, it is necessary to describe the procedure used in solving for the failure load with the aid of Timoshenko's strain energy equation.

The correct mode of failure and its critical load for a given structure can be obtained by assuming as many modes of failure as are deemed necessary to get a mode which gives the lowest critical load. In the table below, cases 1 to 4, are the various modes considered together with their critical loads. The loads presented are the live loads as applied at panel points L_3 of the truss.

Case # 5 is a modification of case #2 , assuming that no lateral restraint was supplied by the center vertical.

CASE	MODE OF FAILURE	CRITICAL LOAD
1		211 kips
2		127 kips
3		112 kips
4		147 kips
5		62 kips

Of these different modes (cases 1 to 4) case #3 yields the lowest critical load. However, these four cases are calculated on the assumption that the loads are applied in the plane of the truss. This means that there is no beam-action on the top chord, and that due to free deflections of the verticals, there is no reduction in the lateral supporting forces.

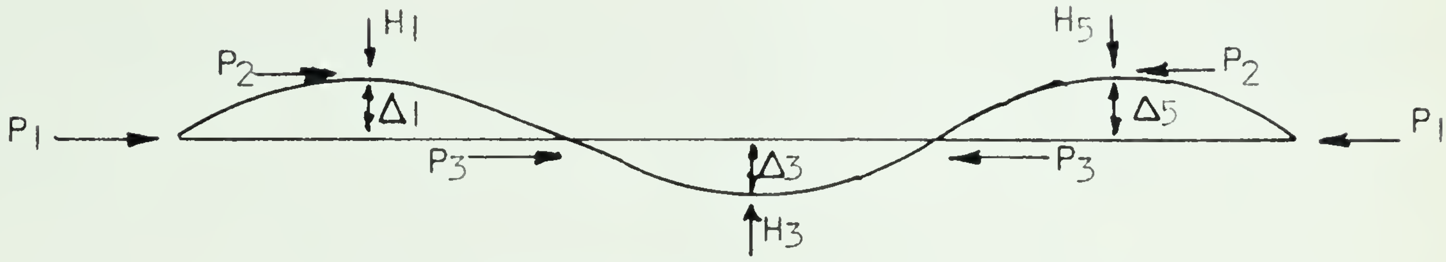
To take into consideration the reduction in the lateral, elastic supports, case #5 was considered. The calculations for this case are based on the consideration that no lateral support was supplied by the centre vertical during buckling. This case gives a mode that most closely resembles the actual deflections of the top chord (see Fig. 20) and gives the lowest critical load.

In order to write the expression for the correct mode of failure, it is necessary to understand the significance of the strain energy produced by the elastic supports and that produced by the bending of the top chord. If the expression for the strain energy due to the elastic supports yields more work than that due to bending of the chord, this signifies that the lateral supports are relatively stiff. This appears to be the situation for this structure, and thus case #3 may not be the exact mode of failure for loading in the planes of the trusses. The expression for case #3 is $y = \delta \sin\left(\frac{4\pi x}{L}\right)$. However, from the examination of the location of the lateral supports, with respect to the mode of failure for case #3, it appears that $y = \delta_1 \sin\left(\frac{4\pi x}{L}\right) + \delta_2 \sin\left(\frac{8\pi x}{L}\right)$ could be a more correct mode of failure.

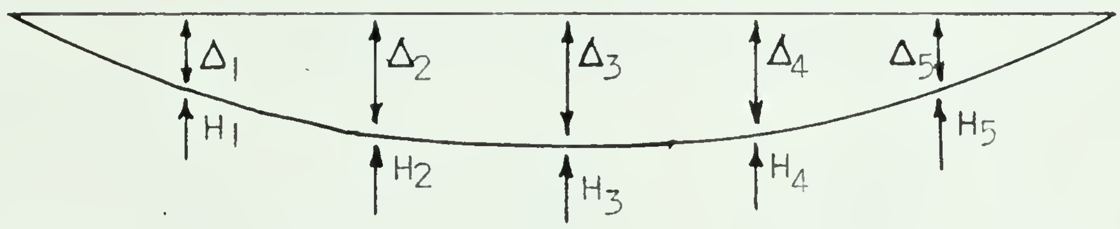
The sample calculation presented is that for case #5.

STRAIN ENERGY EQUATION -- CASE # 5

1. Assume the mode of buckling is a combination of the first and third harmonic, as shown.



$y = \delta_1 \sin(\frac{3\pi x}{L})$, where δ_1 = the amplitude.



$y = \delta_2 \sin(\frac{\pi x}{L})$, where δ_2 = the amplitude.

2. The forces applied to the column are composed of the axial forces P and the lateral forces H.

Where P_1 acts between the limits 0 to L,

P_2 acts between the limits L/6 to 5/6 L,

P_3 acts between the limits L/3 to 2/3L,

and where $H_1 = H_5 = C\,\Delta_1$, $H_2 = H_4 = C\,\Delta_2$, $H_3 = C\,\Delta_3$

3. Assume that the web diagonals do not contribute sufficiently to the spring constant "C", and that it is dependant solely upon the stiffness of the web verticals and the floor beams.

To determine the value of the spring constant, it is necessary to consider the stiffness of a cantilever beam, free at the top of the vertical and fixed at the middle of the floor beam. This spring constant is the load required to deflect the top of the vertical through a distance of one inch. The theoretical value of C is 2.16 kips per inch.

The buckled mode may be expressed as $y = \delta_1 \sin(\frac{3\pi x}{L}) - \delta_2 \sin(\frac{\pi x}{L})$.

The Strain Energy Equation is $W_p = W_s + W_b$, ...(1)

where W_p is the work done by the axial forces P_1, P_2 and P_3,

W_s is the work done by the lateral supports, and

W_b is the work done by the bending of the top chord.

WORK DONE BY THE EXTERNAL FORCES

$W_p = \sum P/2 \int (\frac{dy}{dx})^2 dx$ or,

$$W_p = P_1/2 \int_0^L (\frac{dy}{dx})^2 dx + P_2/2 \int_{L/6}^{5/6 L} (\frac{dy}{dx})^2 dx + P_3/2 \int_{L/3}^{2/3 L} (\frac{dy}{dx})^2 dx \quad \text{........(2)}$$

$$(\frac{dy}{dX})^2 = 9\frac{\pi^2}{L^2}\delta_1^2 \cos^2 \frac{3\pi x}{L} - 6\frac{\pi^2}{L^2}\delta_1 \delta_2 \cos\frac{3\pi x}{L}\cos\frac{\pi x}{L} + \frac{\pi^2}{L^2}\delta_2^2 \cos^2 \frac{\pi x}{L} \quad \text{............(a)}$$

But, $P_1 = (0.697P + 18.1)$ (see Table I) ....(b)

$P_2 = (0.706P + 11.4)$ kips ....(c)

$P_3 = (0.409P + 4.1)$ kips ....(d)

where P is the live load applied to the panel point L_3, and the constant is the force due to the dead load of the structure. The dead load is 5.2 kips per panel point.

By substituting the expressions b, c and d in equation 2, the resulting equation yields,

$$W_p = 0.00334P\delta_2^2 + 0.00625P\delta_1\delta_2 + 0.0304P\delta_1^2 + 0.0695\delta_2^2 + 0.0902\delta_1\delta_2 + 0.629\delta_1^2 \quad \text{.....................(3)}$$

WORK DONE BY THE LATERAL SUPPORTS

$$W_s = C/2(\Delta_1^2 + \Delta_2^2 + \Delta_3^2 + \Delta_4^2 + \Delta_5^2) \quad \ldots\ldots (4)$$

But ,

$$\Delta_1^2 = \Delta_5^2 = 1/4\ \delta_2^2 - \delta_2\delta_1 + \delta_1^2 \quad \ldots\ldots (e)$$

$$\Delta_2^2 = \Delta_4^2 = 3/4\ \delta_2^2 \quad \ldots\ldots (f)$$

$$\Delta_3^2 = \delta_2^2 + 2\delta_2\delta_1 + \delta_1^2 \quad \ldots\ldots (g)$$

Therefore $W_s = 1.5C(\delta_1^2 + \delta_2^2)$ ...... (5)

However, if it be assumed that the centre vertical does no work, then $W_s = C(\delta_2^2 - \delta_2\delta_1 + \delta_1^2)$ ...... (6)

WORK DUE TO BENDING OF THE TOP CHORD

$$W_b = 1/2\ EI\int_0^L \left(\frac{d^2y}{dx^2}\right)^2 dx \quad \ldots\ldots (7)$$

and

$$\left(\frac{d^2y}{dx^2}\right)^2 = 81\frac{\pi^4}{L^4}\delta_1^2 \sin^2\left(\frac{3\pi x}{L}\right) - 18\delta_1\delta_2\frac{\pi^4}{L^4}\sin\left(\frac{\pi x}{L}\right)\sin\left(\frac{3\pi x}{L}\right) + \delta_2^2\frac{\pi^4}{L^4}\sin^2\left(\frac{\pi x}{L}\right) \quad \ldots\ldots (h)$$

By substituting expression (h) in equation (7) , the resulting equation yields $W_b = 1/2EI\left(\frac{81\pi^4}{2L^3}\delta_1^2 + \frac{\pi^4}{2L^3}\delta_2^2\right)$ ...... (8)

By substituting equations (3), (6) and (8) into equation (1), and by substituting C= 2.16 kip/in, and L= 960 in, the Strain Energy Equation yields

$$P(0.334\delta_2^2 + 0.625\delta_1\delta_2 + 3.040\delta_1^2) = 210.625\delta_2^2 + 280.93\delta_1^2 - 225.02\delta_1\delta_2 \quad \ldots\ldots (9)$$

Equation (9) cannot be solved directly. It is necessary to assume some relation between δ_1 and δ_2. Then P can be solved, because the (δ) terms will cancel out.

To determine Pcr, the following five different relations between δ_1 and δ_2 were considered.

$\delta_2/\delta_1 = 0$ P= 92.3 kips

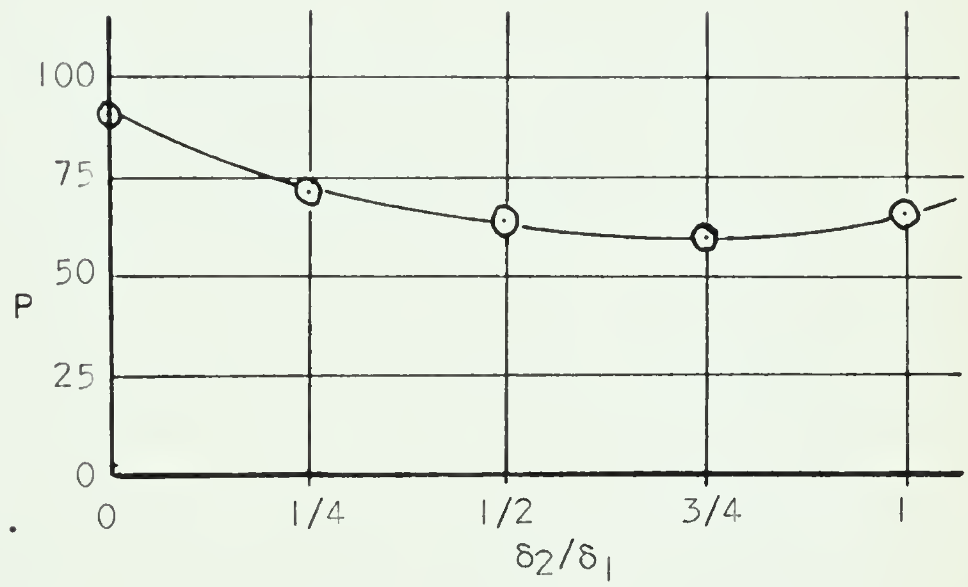
$\delta_2/\delta_1 = 1/4$ P= 73.6 kips

$\delta_2/\delta_1 = 1/2$ P= 65.0 kips

$\delta_2/\delta_1 = 3/4$ P= 62.0 kips

$\delta_2/\delta_1 = 1$ P= 66.6 kips

From the plot of "P" v.s. "δ_2/δ_1", for all practical purposes, $\delta_2/\delta_1 = 3/4$ yields the lowest critical load. Therefore Pcr = 62.0 kips .



SOUTHWELL'S THEORY

Southwell's Theory, as described in the Historical Review, applies readily to a simple column. However, to apply his theory to the buckling of the top chords of the pony-truss bridge, the inflection points had to be determined.

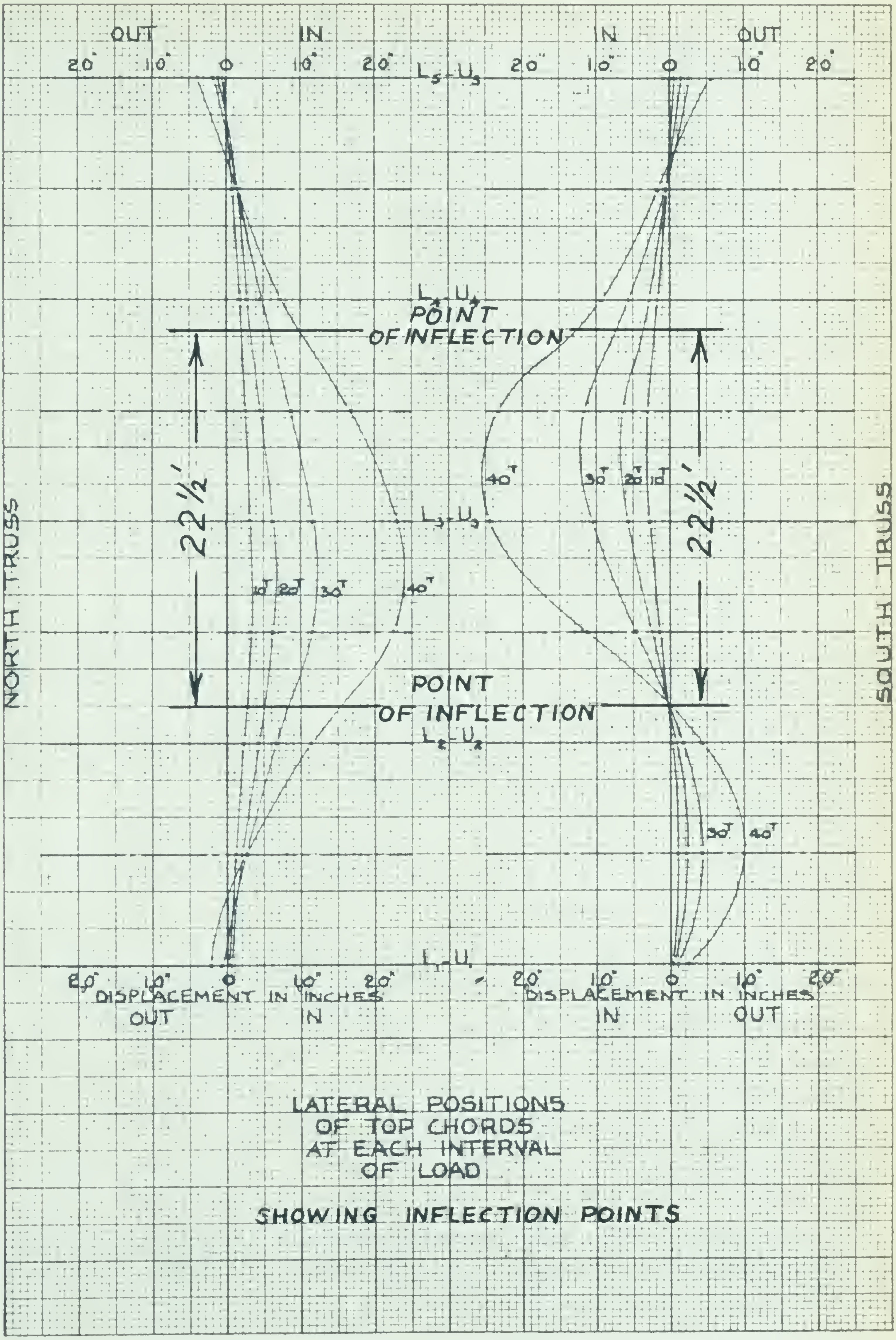
The procedure used, is as follows:

1. Locate the points of inflection (see Fig. 21).

2. Determine the maximum deflection Δ between the points of inflection (see Fig. 22).

3. Plot Δ/P vs. Δ where P is the axial load on the simple column.

4. The slope of the best fitted straight line is $1/P_{cr}$, the reciprocal of the critical load. (see Fig. 23 and 24).



OUT
IN
IN
OUT
20"
10"
0
10"
20"
L_5-U_5
20"
10"
0
10"
20"
L_4-U_4
POINT
OF INFLECTION
22 1/2'
22 1/2'
40T
30T
20T
10T
L_3-U_3
10T
20T
30T
40T
NORTH TRUSS
SOUTH TRUSS
POINT
OF INFLECTION
L_2-U_2
30T
40T
L_1-U_1
20"
10"
0
10"
20"
DISPLACEMENT IN INCHES
OUT
IN
20"
10"
0
10"
20"
DISPLACEMENT IN INCHES
IN
OUT
LATERAL POSITIONS
OF TOP CHORDS
AT EACH INTERVAL
OF LOAD
SHOWING INFLECTION POINTS

Fig. 21

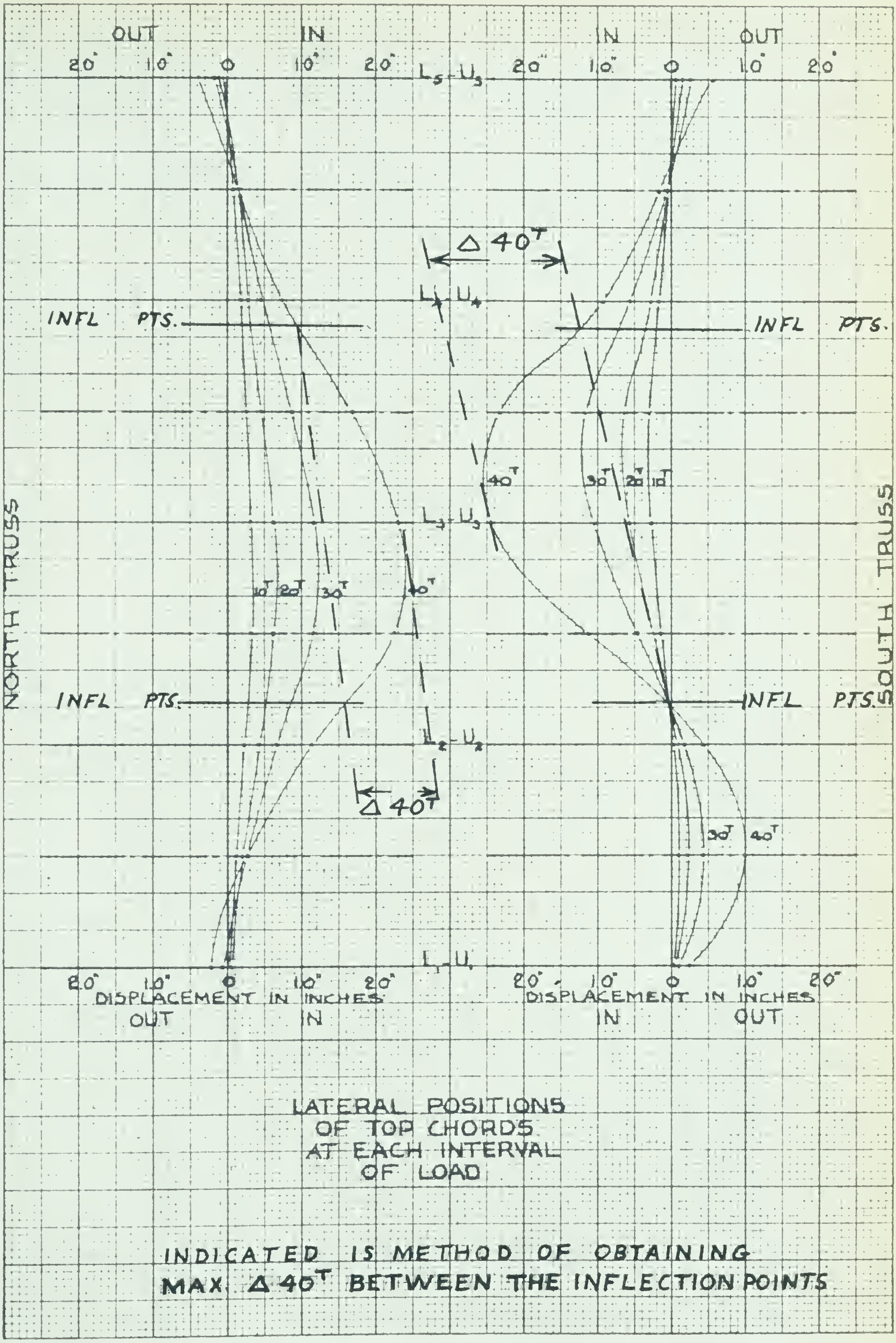
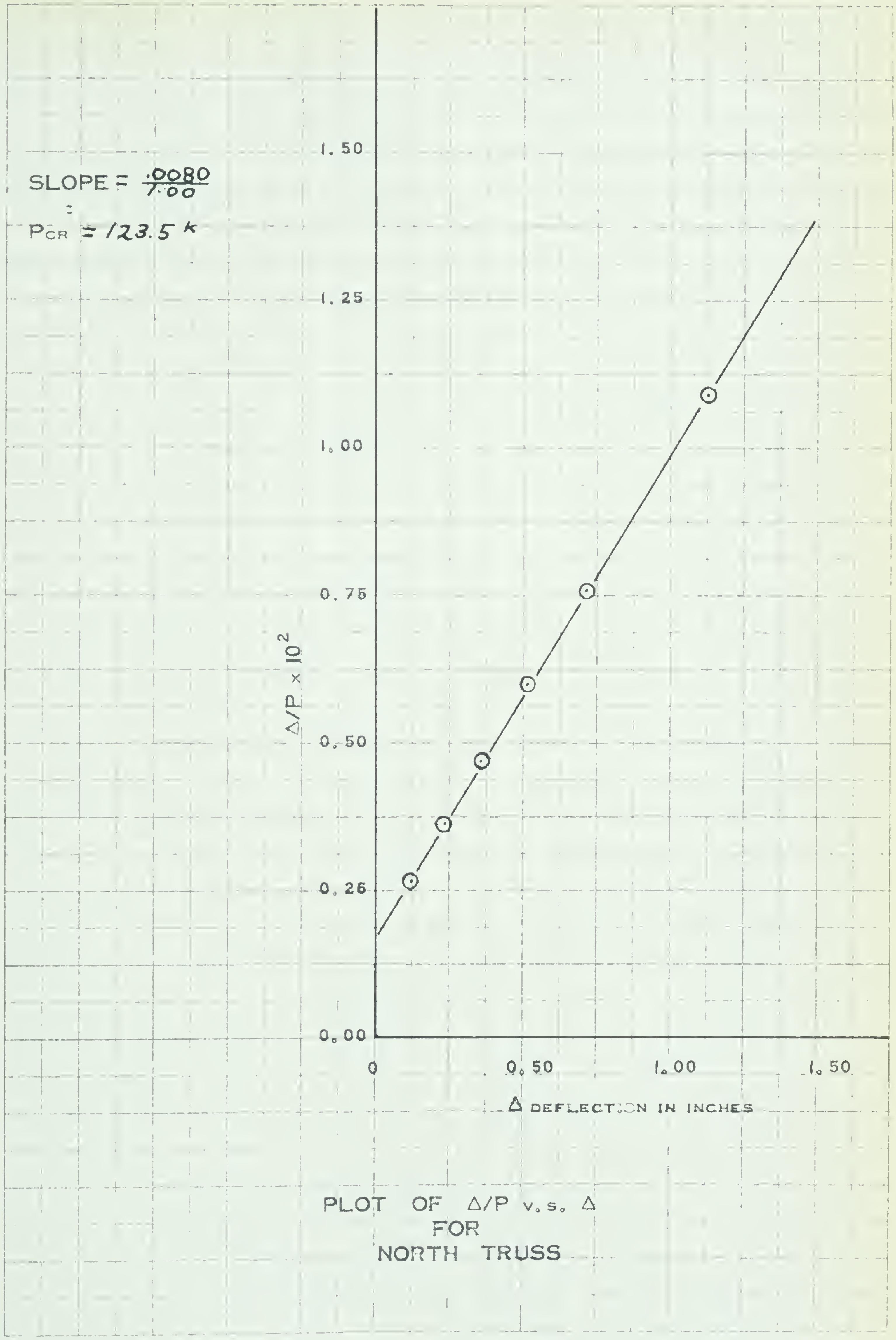

OUT
IN
IN
OUT
20" 10" 0 10" 20"
L_5-U_5
20" 10" 0 10" 20"
Δ 40ᵀ
L_4-U_4
INFL PTS.
INFL PTS.
40ᵀ
30ᵀ 20ᵀ 10ᵀ
NORTH TRUSS
L_3-U_3
SOUTH TRUSS
10ᵀ 20ᵀ 30ᵀ 40ᵀ
INFL PTS.
INFL PTS.
L_2-U_2
Δ 40ᵀ
30ᵀ 40ᵀ
L_1-U_1
20" 10" 0 10" 20"
DISPLACEMENT IN INCHES
OUT IN
20" 10" 0 10" 20"
DISPLACEMENT IN INCHES
IN OUT
LATERAL POSITIONS
OF TOP CHORDS
AT EACH INTERVAL
OF LOAD
INDICATED IS METHOD OF OBTAINING
MAX Δ 40ᵀ BETWEEN THE INFLECTION POINTS

Fig. 22



SLOPE = .0080 / 1.00
PCR = 123.5 k
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Δ/P x 10^2
0
0.50
1.00
1.50
Δ DEFLECTION IN INCHES
PLOT OF Δ/P v.s. Δ
FOR
NORTH TRUSS

Fig. 23

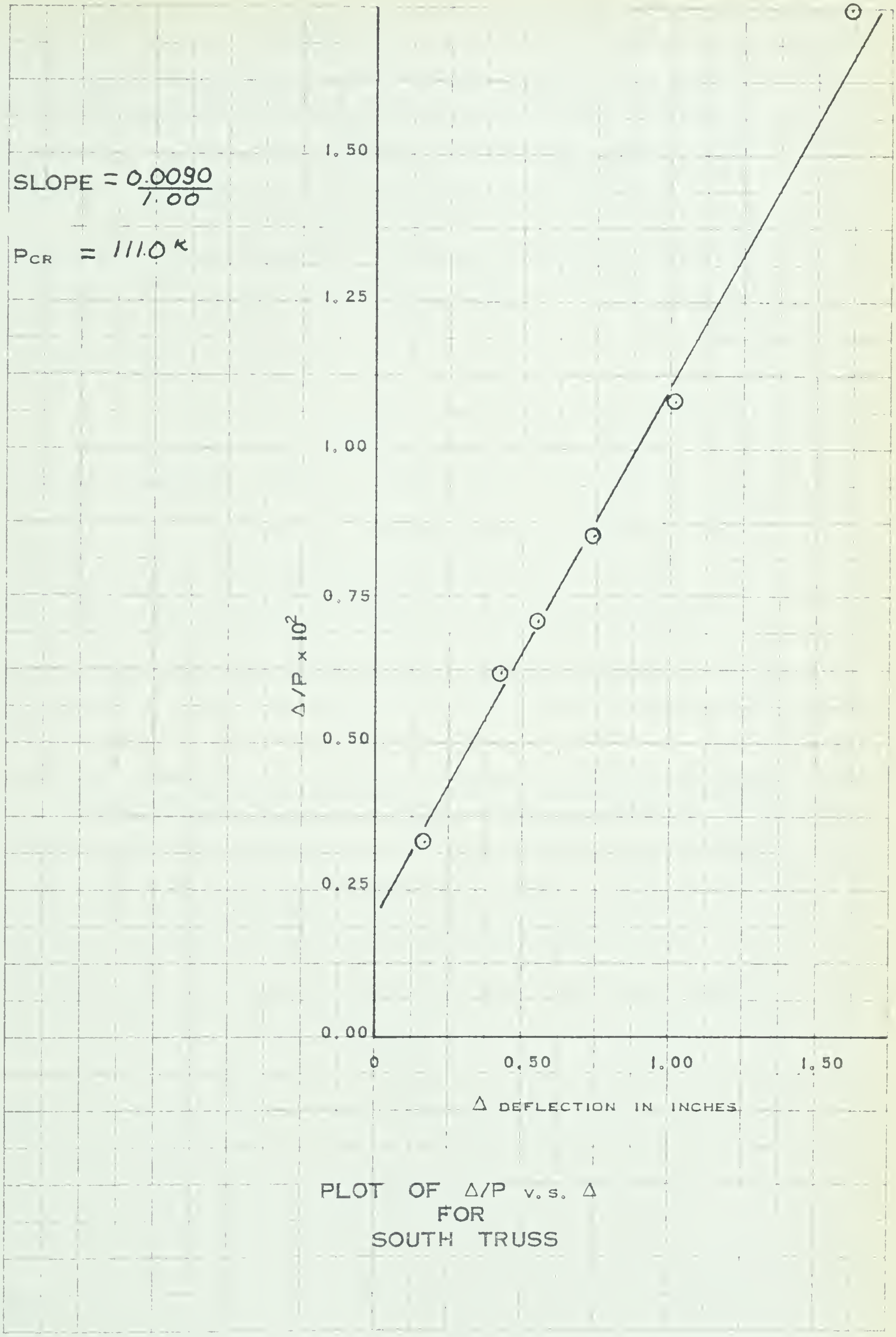

SLOPE = 0.0090 / 1.00
PCR = 111.0 K
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Δ/P x 10²
0
0.50
1.00
1.50
Δ DEFLECTION IN INCHES
PLOT OF Δ/P v.s. Δ
FOR
SOUTH TRUSS

Fig. 24

APPENDIX II

DATA

The original "SR-4" strain gauge data is not contained in this thesis. However, the original readings and a copy of the breakdown of these readings are available, in the Department of Civil Engineering, the University of Alberta. Another copy, of the breakdown of the "SR-4" strain gauge, data is available at the Research Council of Alberta.

This data must be corrected due to the drift in the "SR-4" strain indicator, before it can be used. For all the graphs, Fig. 25 to 44, the proper corrections have been applied. The corrections have been obtained from the plot on Fig. 50. This plot shows the change in the "SR-4" strain gauge readings for each load interval due to the voltage change of the power supply.

This correction curve of Fig. 50 was obtained by plotting the recorded strain for gauges U, V, J and K of the floor beams with the exception of the centre floor beam, Fig. 13. These gauges were positioned vertically on the neutral axis of the floor beams. Thus no strain should have occurred at these gauges.

This plot clearly shows the high drift or change in the datum of the "SR-4" strain gauge readings when the power supply voltage is drastically changed. A new battery was installed prior to 20 ton readings.

DATA

LATERAL DEFLECTIONS OF THE TOP CHORD

	LOCATION	Deflection In Inches For Each Load Interval					
		10 Tons	20 Tons	25 Tons	30 Tons	35 Tons	40 Tons
SOUTH TRUSS	U_1	+0.010	+0.015	+0.023	+0.049	+0.084	+0.240
	$U_1 - U_2$	+0.103	+0.222	+0.309	+0.418	+0.563	+1.011
	U_2	+0.033	+0.087	+0.124	+0.161	+0.211	+0.425
	$U_2 - U_3$	-0.113	-0.226	-0.340	-0.483	-0.697	-1.110
	U_3	-0.268	-0.563	-0.819	-1.125	-1.565	-2.462
	$U_3 - U_4$	-0.290	-0.621	-0.883	-1.178	-1.588	-2.321
	U_4	-0.171	-0.268	-0.494	-0.531	-0.702	-0.936
	$U_4 - U_5$	-0.028	-0.058	-0.060	-0.055	-0.029	-0.148
	U_5	+0.063	+0.141	+0.201	+0.278	+0.390	+0.584
NORTH TRUSS	U_1	-0.013	-0.001	+0.021	+0.056	+0.106	+0.206
	$U_1 - U_2$	-0.013	-0.176	-0.208	-0.240	-0.257	-0.224
	U_2	-0.213	-0.411	-0.543	-0.684	-0.847	-1.117
	$U_2 - U_3$	-0.313	-0.629	-0.863	-1.139	-1.500	-2.250
	U_3	-0.313	-0.624	-0.872	-1.172	-1.581	-2.310
	$U_3 - U_4$	-0.240	-0.470	-0.650	-0.863	-1.142	-1.686
	U_4	-0.159	-0.287	-0.381	-0.476	-0.595	-0.741
	$U_4 - U_5$	-0.071	-0.144	-0.161	-0.174	-0.170	-0.114
	U_5	+0.031	+0.087	+0.131	+0.181	+0.262	+0.398

NOTE: Positive readings are deflections outward, and negative readings are deflections inward. Locations such as $U_2 - U_3$ are midway between panel points U_2 and U_3.

Table 2

DATA

VERTICAL DEFLECTIONS OF THE FLOOR BEAMS IN CENTIMETERS

LOAD	10 Tons	20 Tons	25 Tons	30 Tons	35 Tons	40 Tons
Location			Deflections in Centimeters			
L_{1S}	0.35	0.60	0.80	0.80	1.00	1.20
L_1	0.25	0.50	0.60	0.75	0.90	1.10
L_{1N}	0.25	0.45	0.60	0.75	0.95	1.10
L_{2S}	0.55	1.10	1.40	1.55	1.90	2.10
L_2	0.50	1.10	1.40	1.65	1.95	1.50
L_{2N}	0.45	0.90	1.00	1.40	1.80	2.25
L_{3S}	0.75	1.35	1.95	2.20	2.75	3.25
L_3	1.00	2.15	3.00	3.65	4.70	6.25
L_{3N}	0.75	1.20	1.65	2.20	2.65	3.30
L_{4S}	0.30	0.90	1.10	1.35	1.70	2.10
L_4	0.50	1.05	1.35	1.65	2.05	2.50
L_{4N}	0.40	0.55	1.05	1.50	1.80	2.10
L_{5S}	0.25	0.55	0.70	0.85	1.05	1.30
L_5	0.25	0.50	0.65	0.80	0.95	1.10
L_{5N}	0.25	0.45	0.65	0.80	0.95	1.15

Note: Location such as L_{1S} means the south end of the floor beam at panel point 1, L_1 means the middle of the floor beam and L_{1N} means the north end of the floor beam.

Table 3

DATA

ANGLE CHANGES BETWEEN THE VERTICALS AND THE FLOOR BEAMS

	LOAD	10 Tons	20 Tons	25 Tons	30 Tons	35 Tons	40 Tons
	Location	Deflections in Inches					
SOUTH TRUSS	L_1	+0.0003	+0.0008	+0.0009	+0.0009	+0.0005	-0.0004
	L_2	-0.0034	-0.0073	-0.0095	-0.0118	-0.0150	-0.0213
	L_3	+0.0058	+0.0066	+0.0076	+0.0095	+0.0114	+0.0193
	L_4	+0.0013	+0.0052	+0.0048	+0.0066	+0.0091	+0.0138
	L_5	-0.0009	-0.0021	-0.0032	-0.0047	-0.0067	-0.0117
NORTH TRUSS	L_1	-0.0002	-0.0007	-0.0013	-0.0022	-0.0036	-0.0065
	L_2	+0.0006	+0.0014	+0.0024	+0.0036	+0.0052	+0.0071
	L_3	-0.0058	-0.0105	-0.0125	-0.0144	-0.0163	-0.0165
	L_4	-0.0004	-0.0008	-0.0008	-0.0005	-0.0001	+0.0002
	L_5	-0.0014	-0.0030	-0.0041	-0.0055	-0.0074	-0.0108

Note: Negative readings are deflections to the inside of the bridge. To obtain the angle changes in radians, divide the deflection readings by 19.25 inches.

Table 4

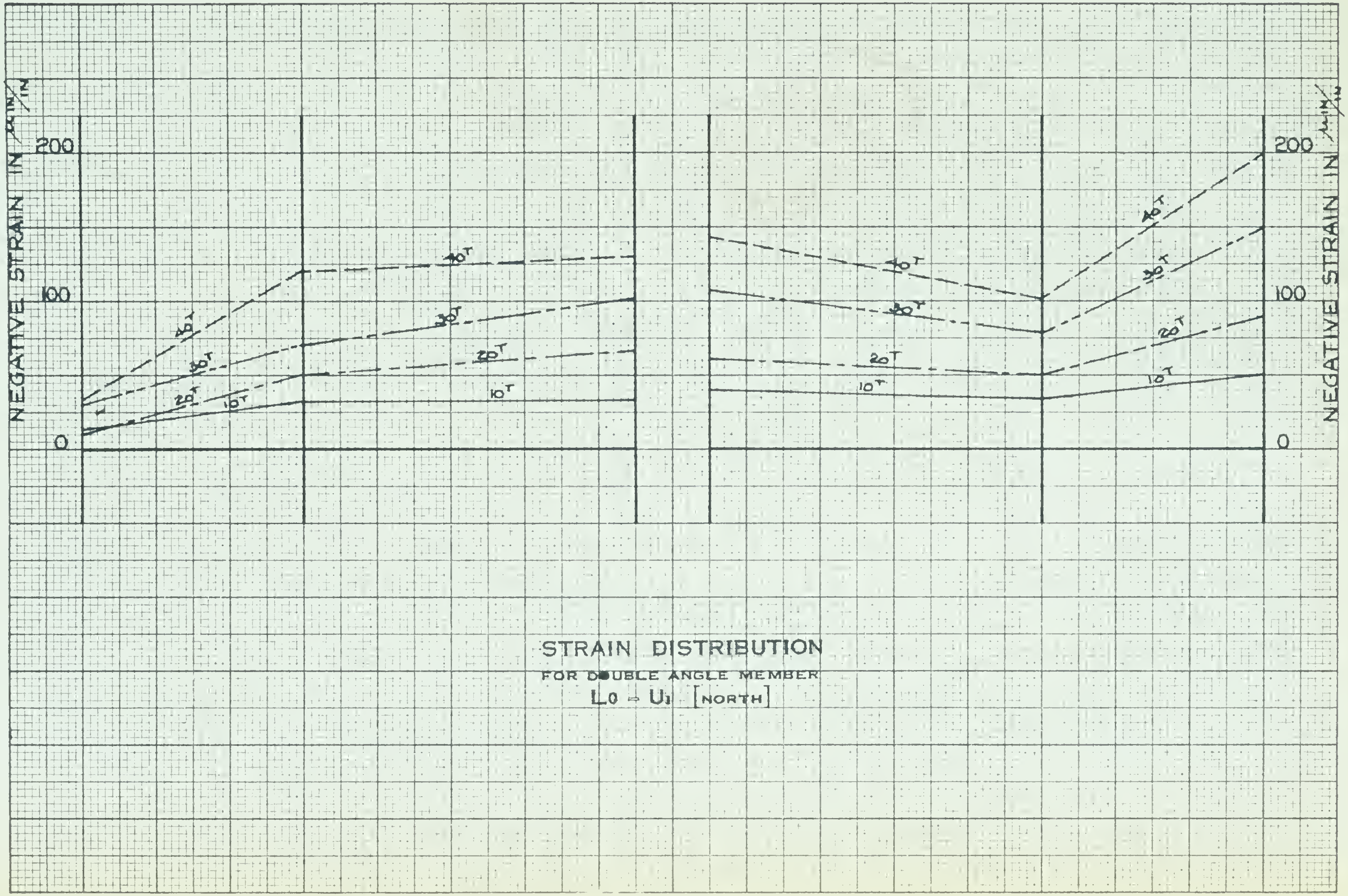

NEGATIVE STRAIN IN μIN/IN
200
100
0
40T
30T
20T
10T
STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
L0 - U1 [NORTH]

Fig. 25

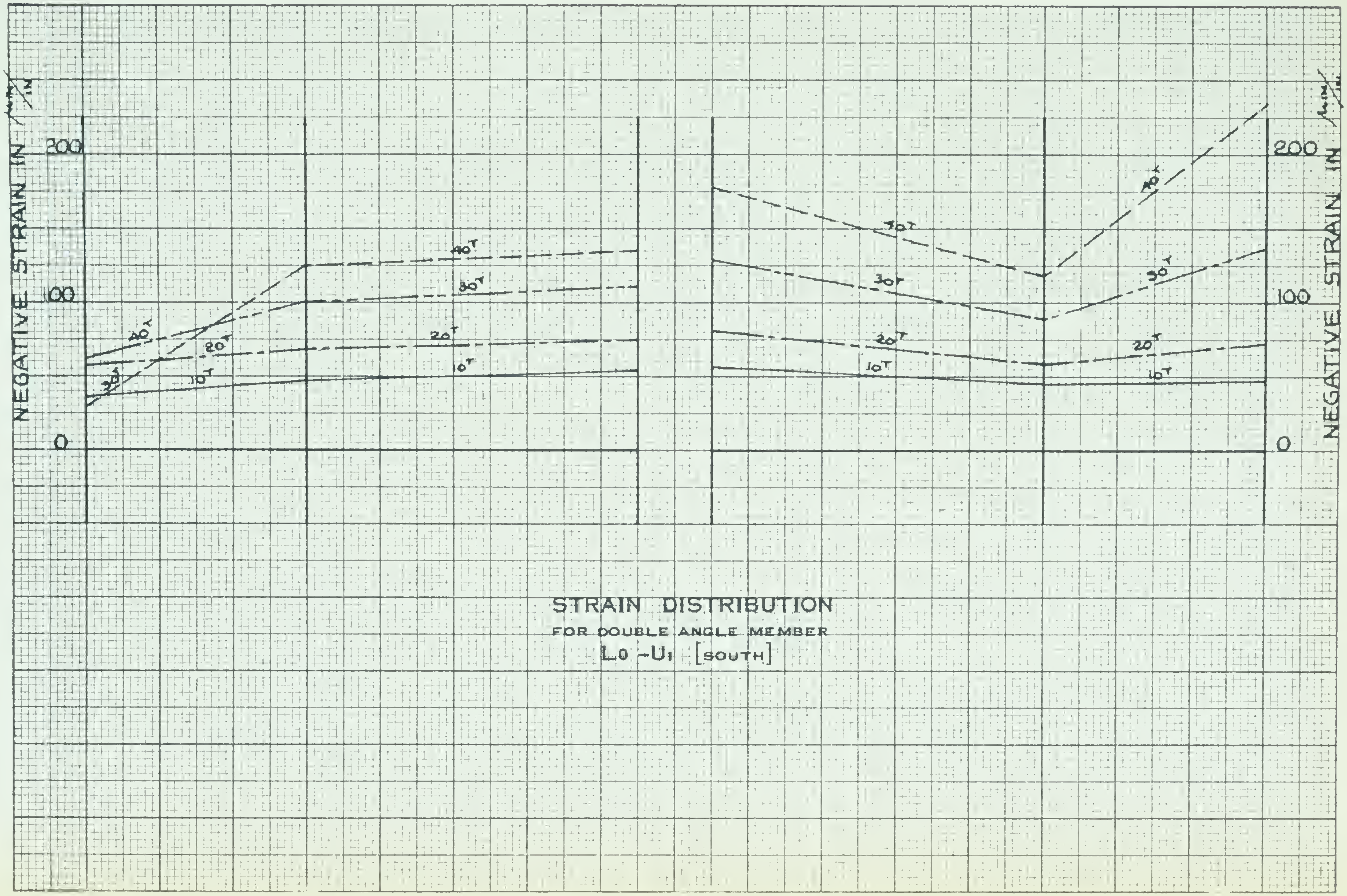

NEGATIVE STRAIN IN μ in/in
200
100
0
40T
30T
20T
10T
STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
Lo -U1 [SOUTH]

Fig. 26

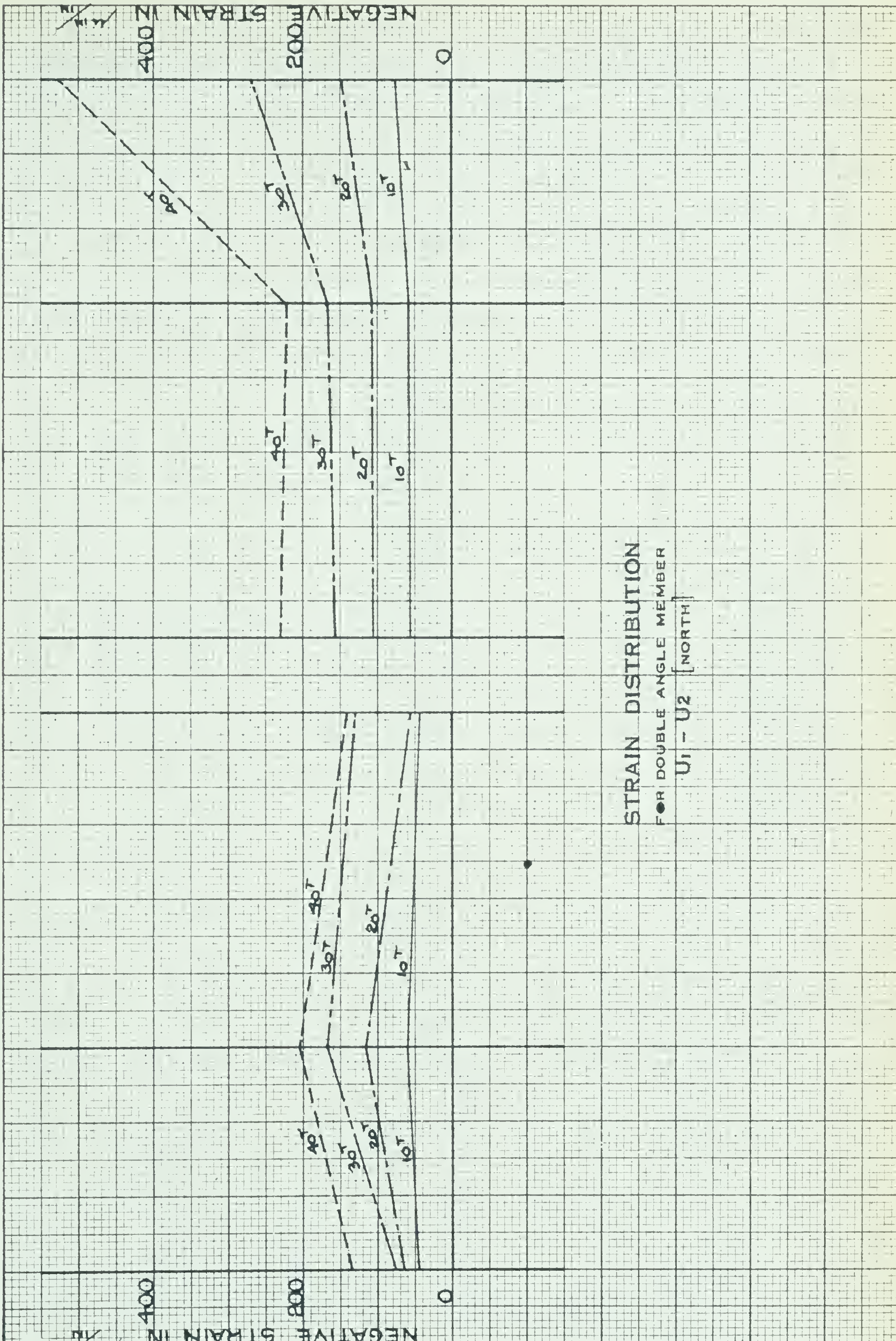

NEGATIVE STRAIN IN
400
200
0
40T
30T
20T
10T
STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
U1 - U2 [NORTH]

Fig. 27

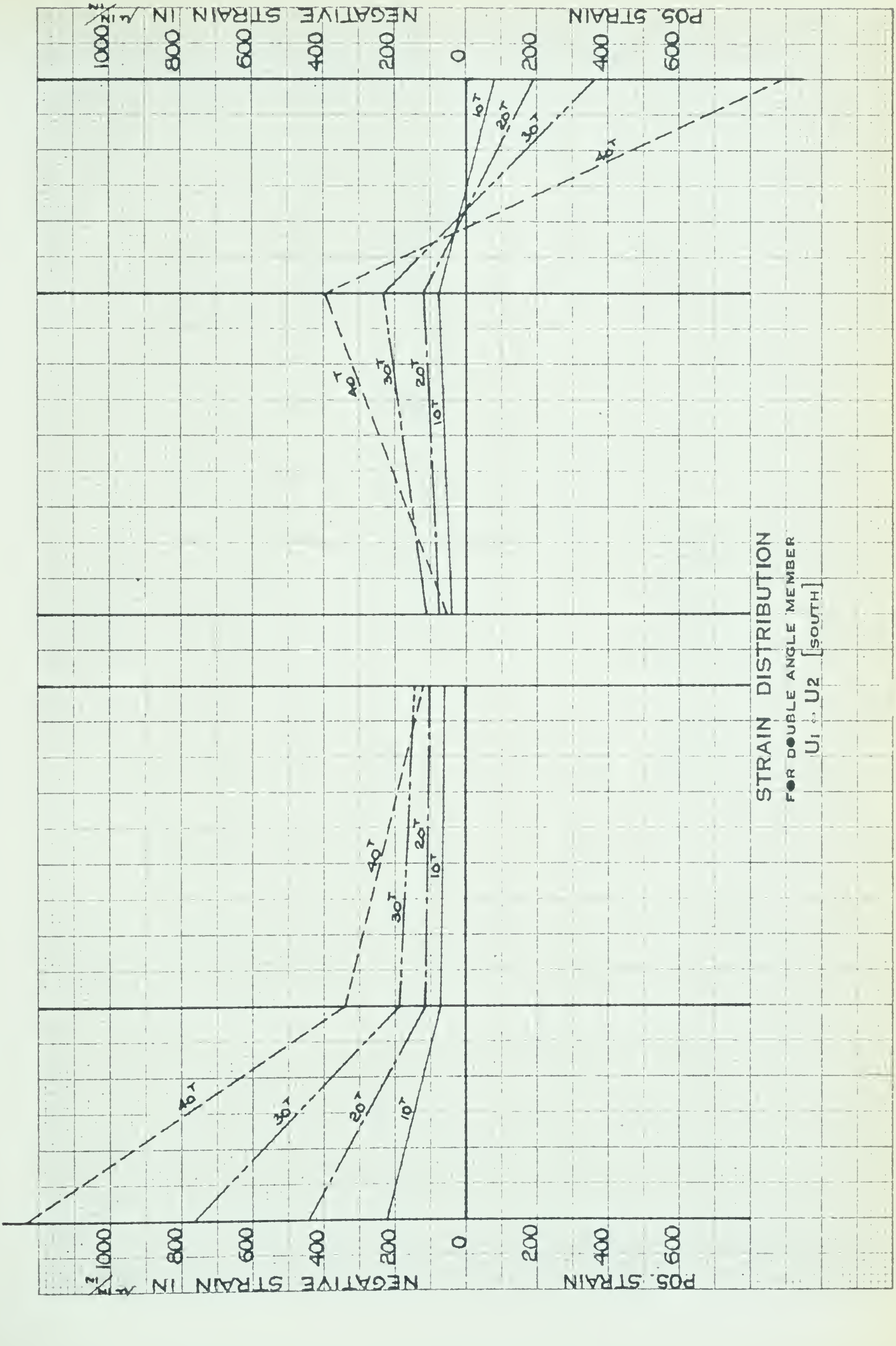

STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
U1 – U2 [SOUTH]
NEGATIVE STRAIN IN
POS. STRAIN
1000
800
600
400
200
0
200
400
600
10T
20T
30T
40T

Fig. 28

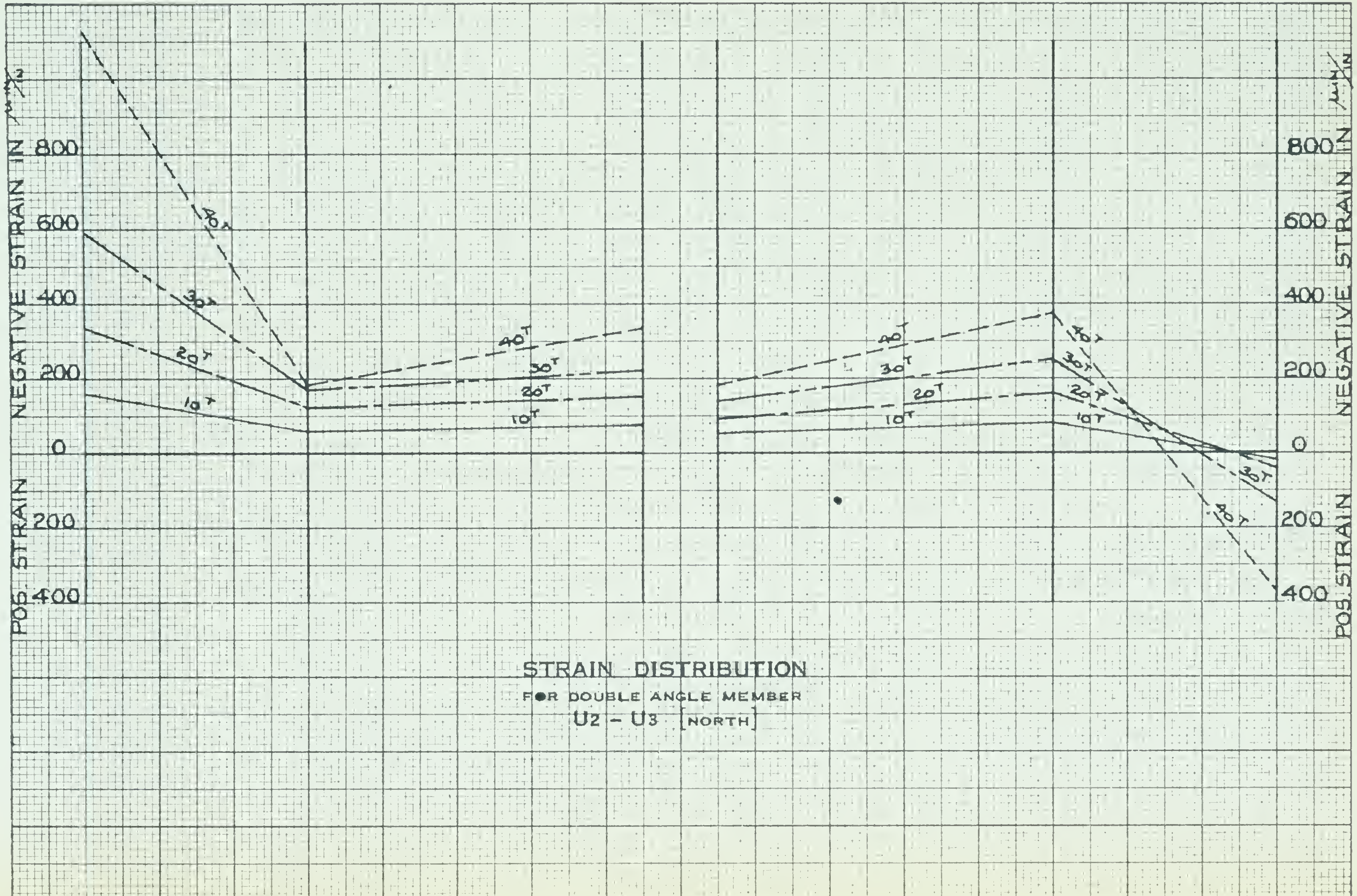

NEGATIVE STRAIN IN μin/in
POS. STRAIN
800
600
400
200
0
200
400
40T
30T
20T
10T
STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
U2 – U3 [NORTH]

Fig. 29

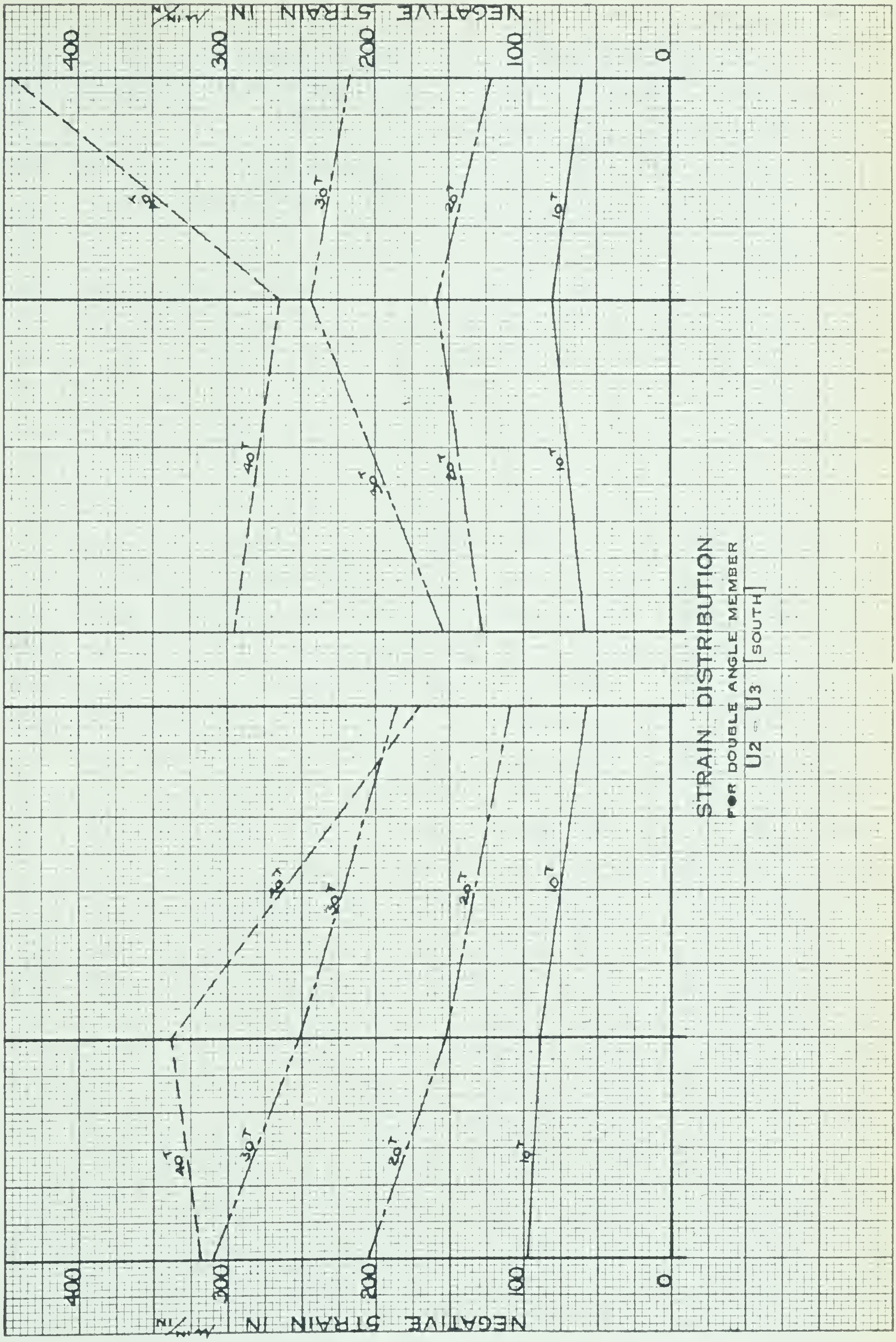

STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
$U_2 - U_3$ [SOUTH]
NEGATIVE STRAIN IN
0
100
200
300
400
10^T
20^T
30^T
40^T

Fig. 30

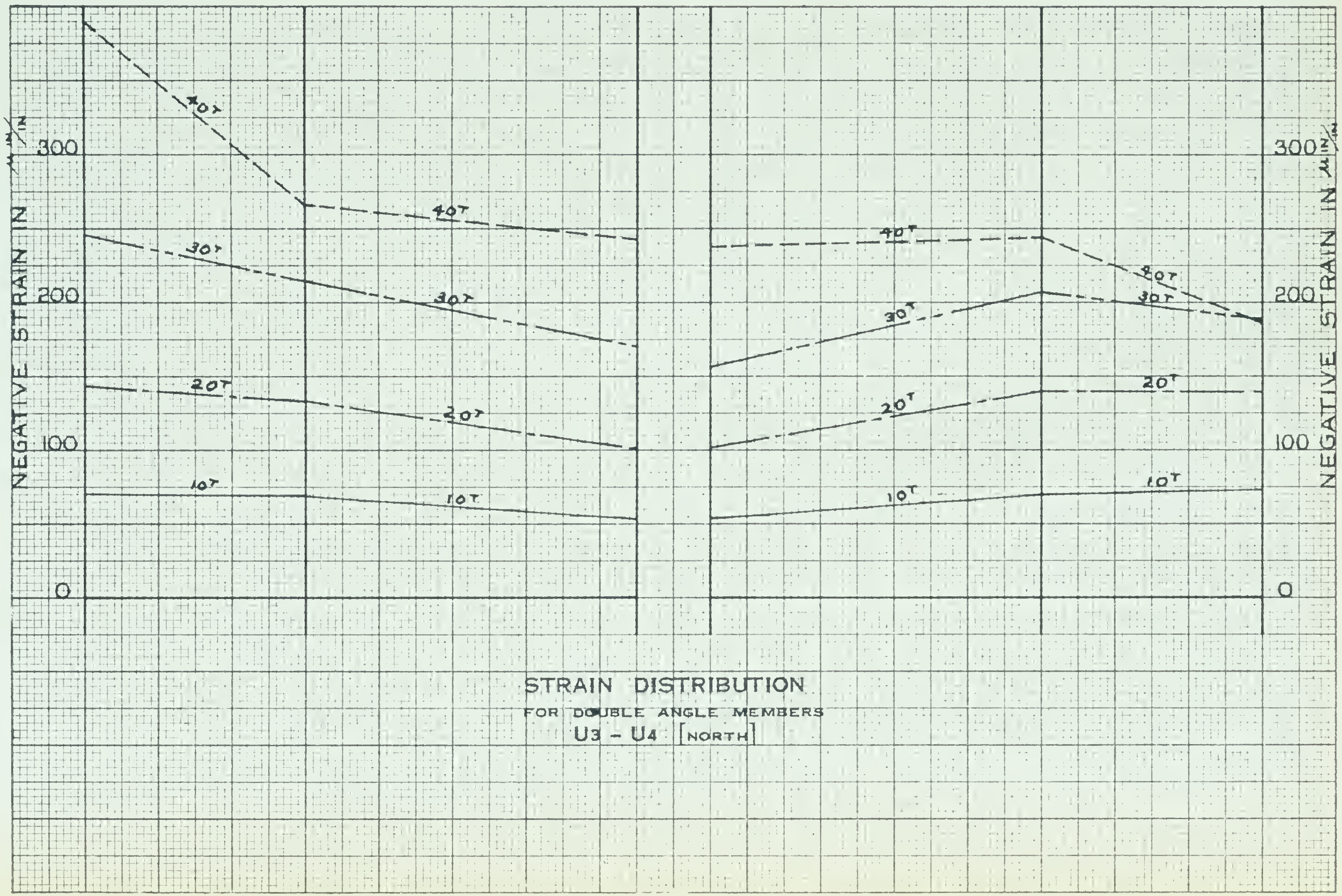

NEGATIVE STRAIN IN µ in/in
300
200
100
0
40T
30T
20T
10T
STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBERS
U3 - U4 [NORTH]

Fig. 31

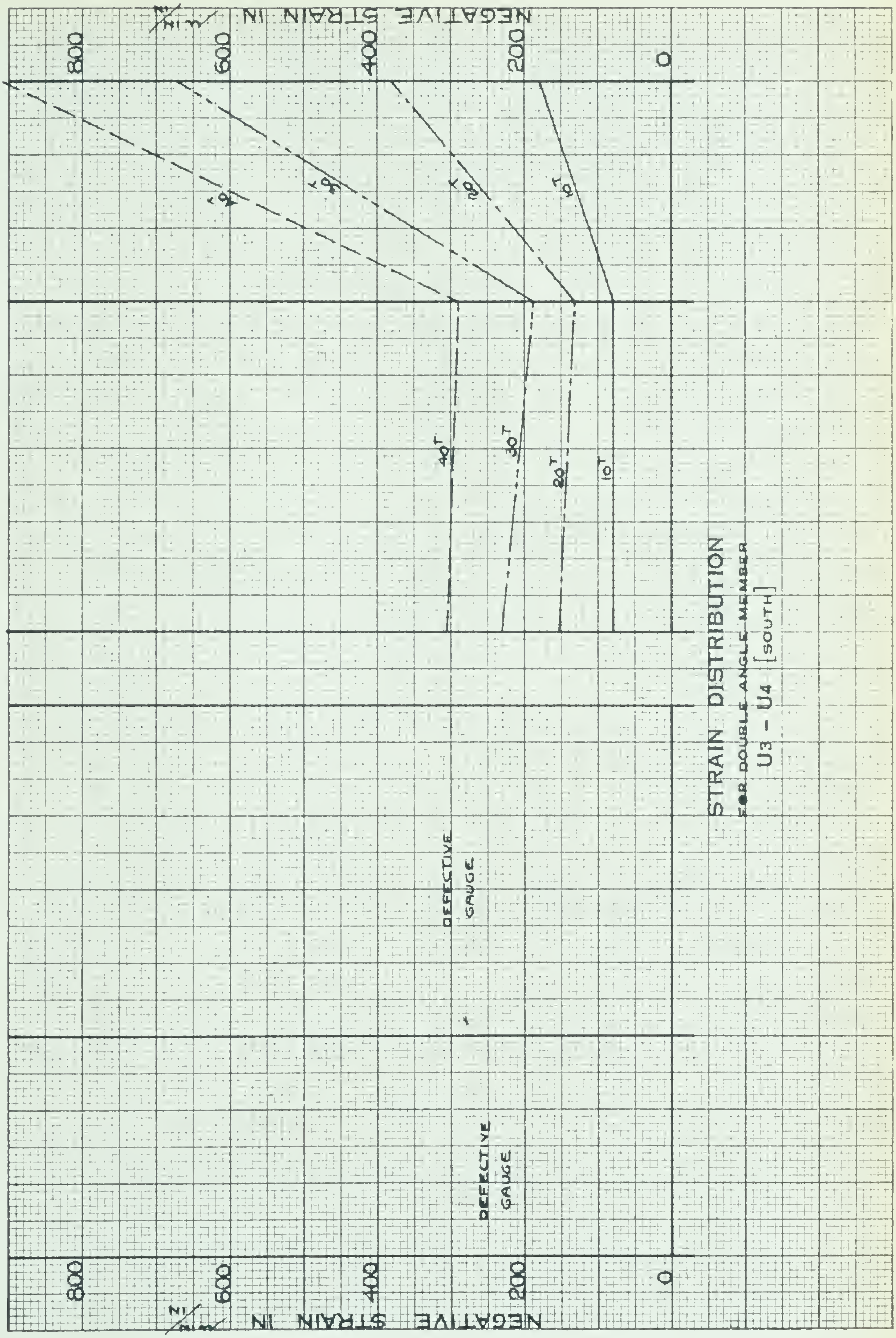

NEGATIVE STRAIN IN
800
600
400
200
0
40T
30T
20T
10T
DEFECTIVE GAUGE
DEFECTIVE GAUGE
STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
U3 - U4 [SOUTH]

Fig. 32

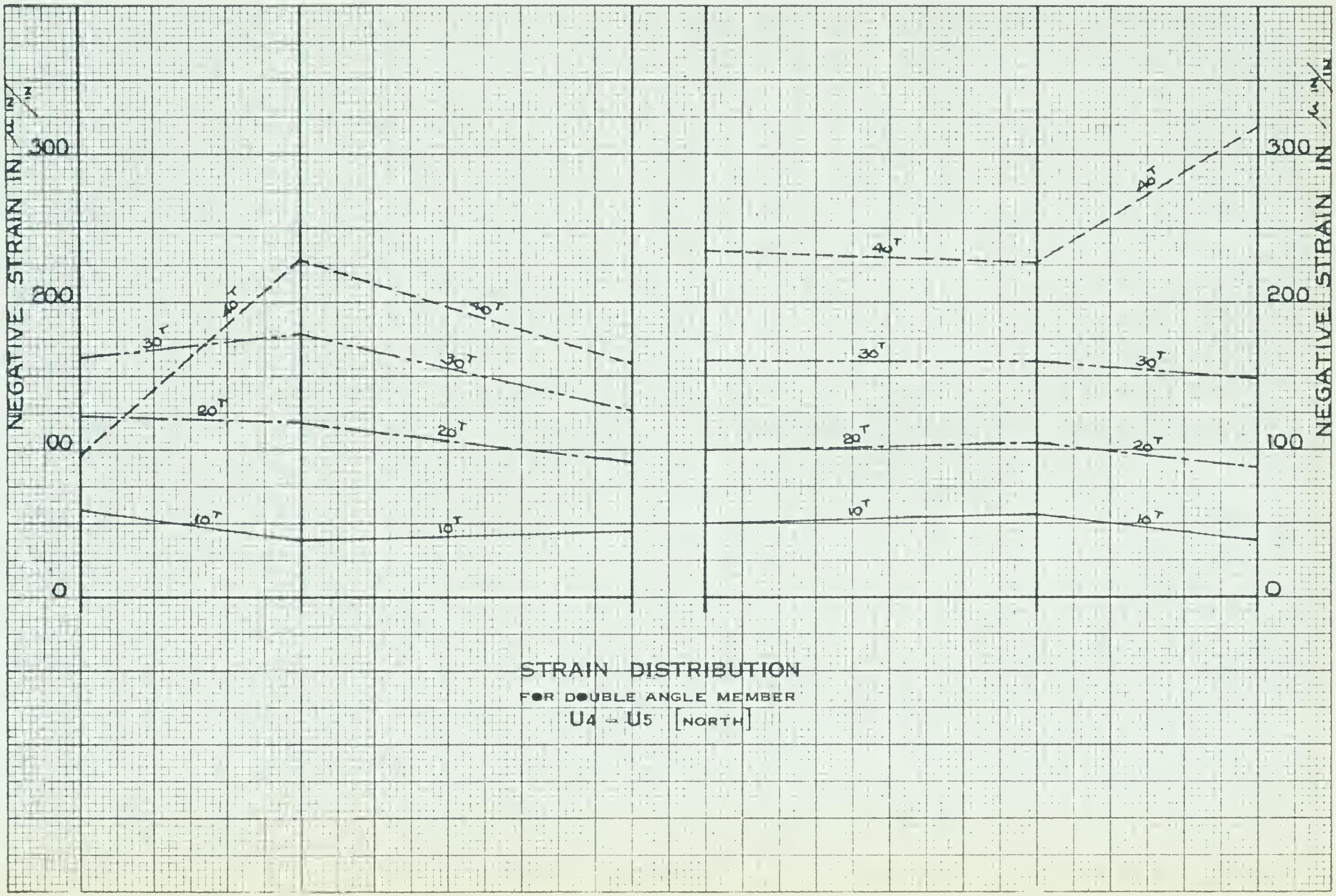

NEGATIVE STRAIN IN μ IN/IN
300
200
100
0
40T
30T
20T
10T
STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
U4 – U5 [NORTH]

Fig. 33

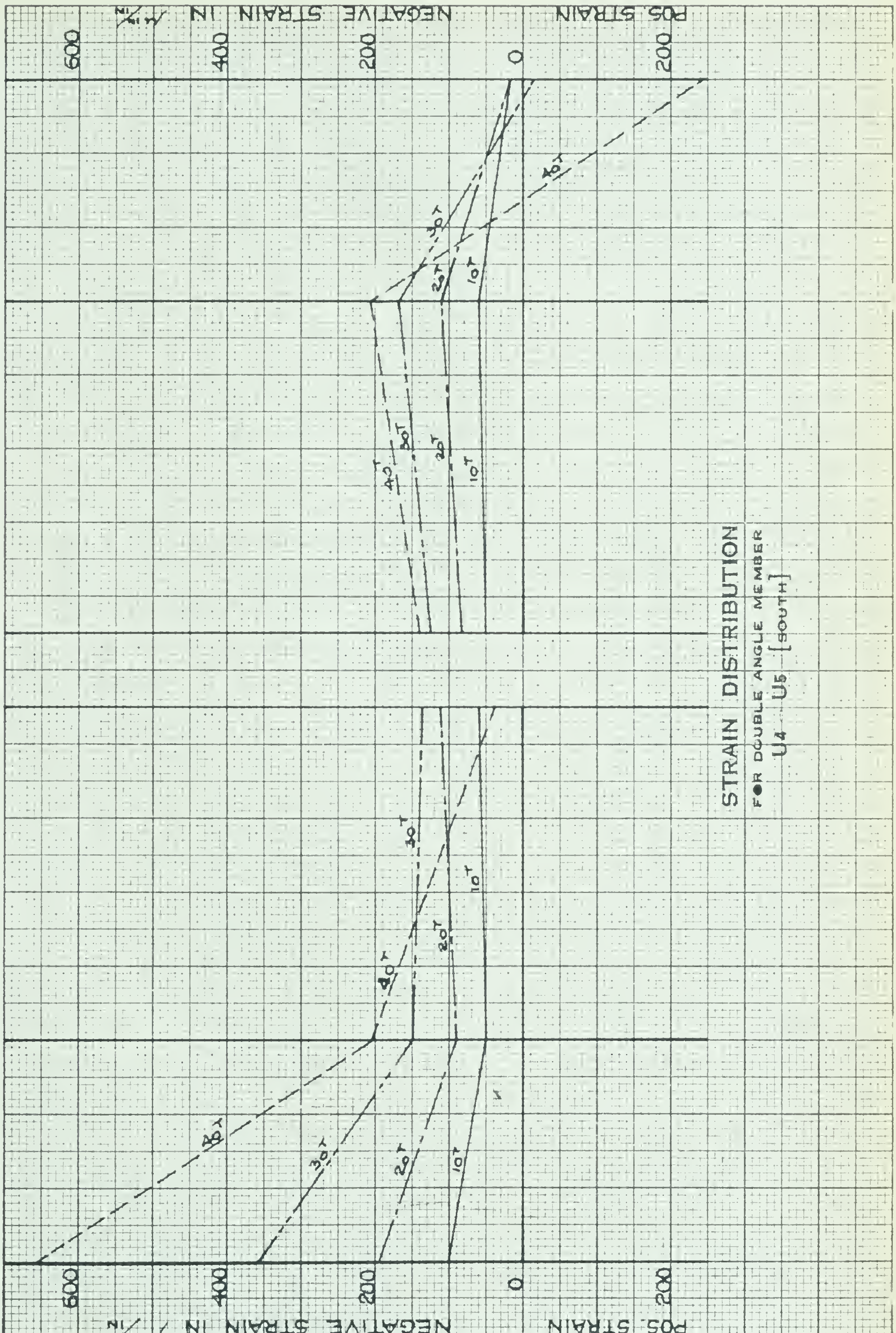
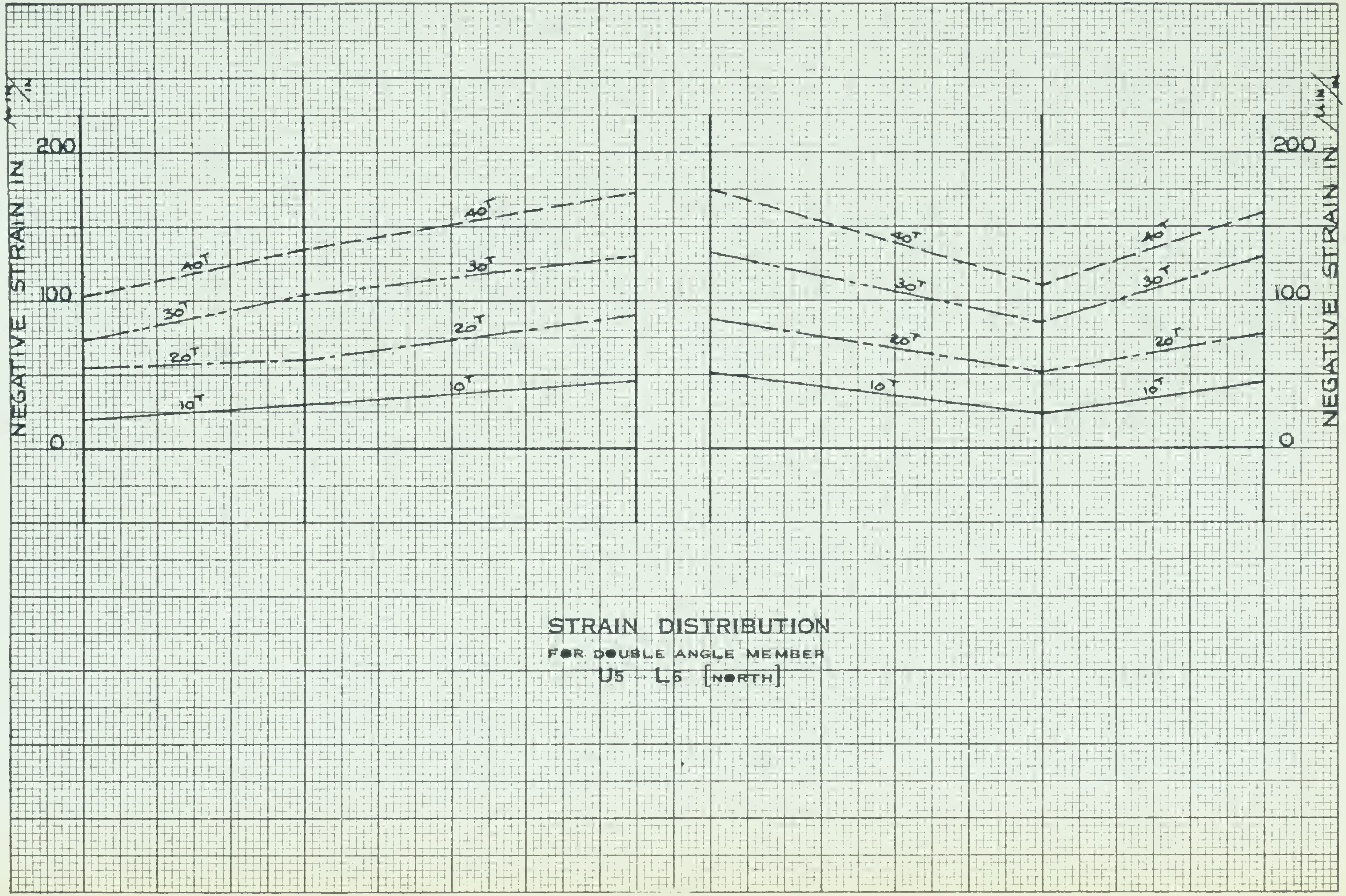

STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
U4 U5 [SOUTH]
NEGATIVE STRAIN IN IN/IN
POS STRAIN
600
400
200
0
200
10T
20T
30T
40T

Fig. 34



STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
U5 – L6 [NORTH]

Fig. 35

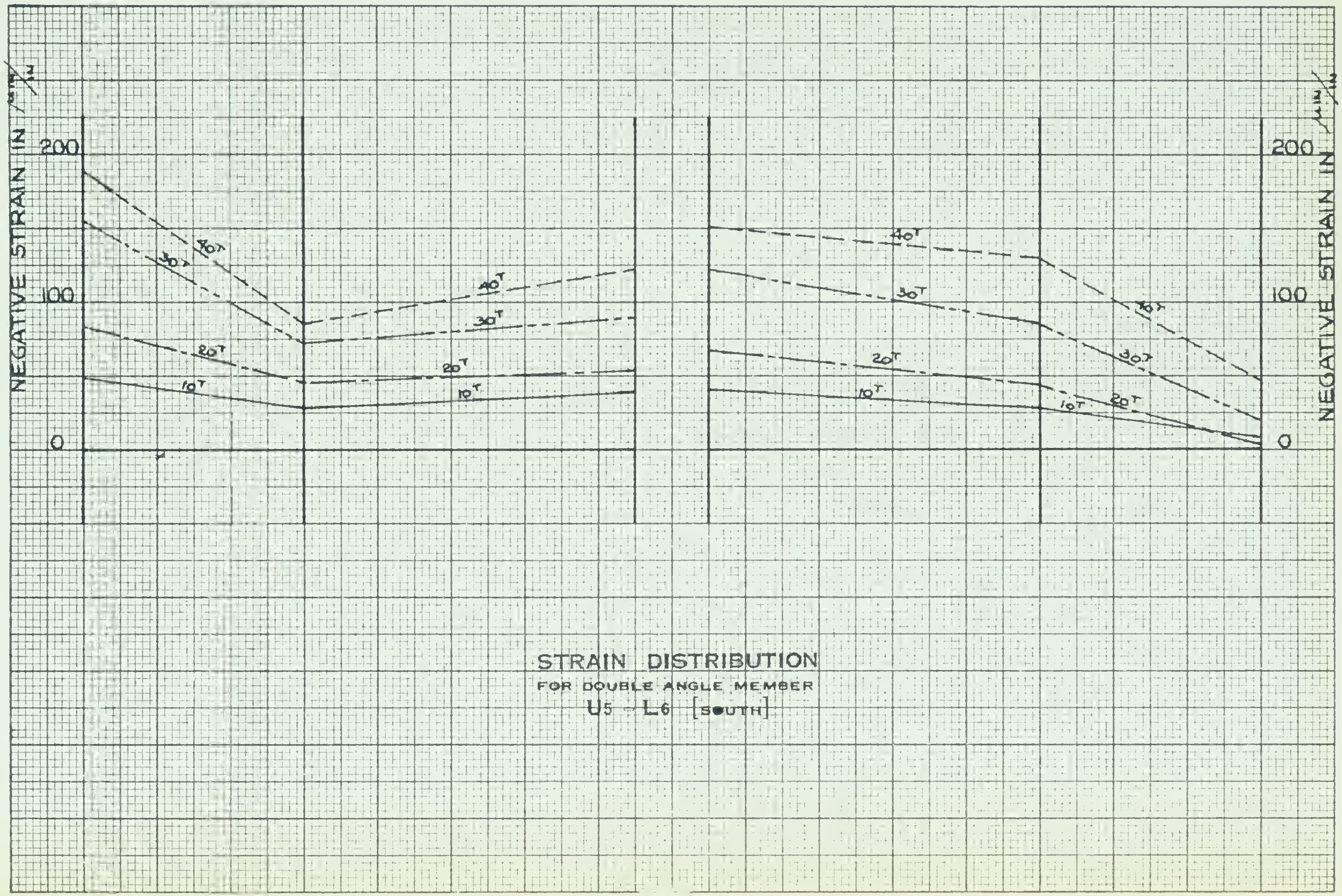

NEGATIVE STRAIN IN μIN/IN
200
100
0
40T
30T
20T
10T
STRAIN DISTRIBUTION
FOR DOUBLE ANGLE MEMBER
U5 - L6 [SOUTH]

Fig. 36

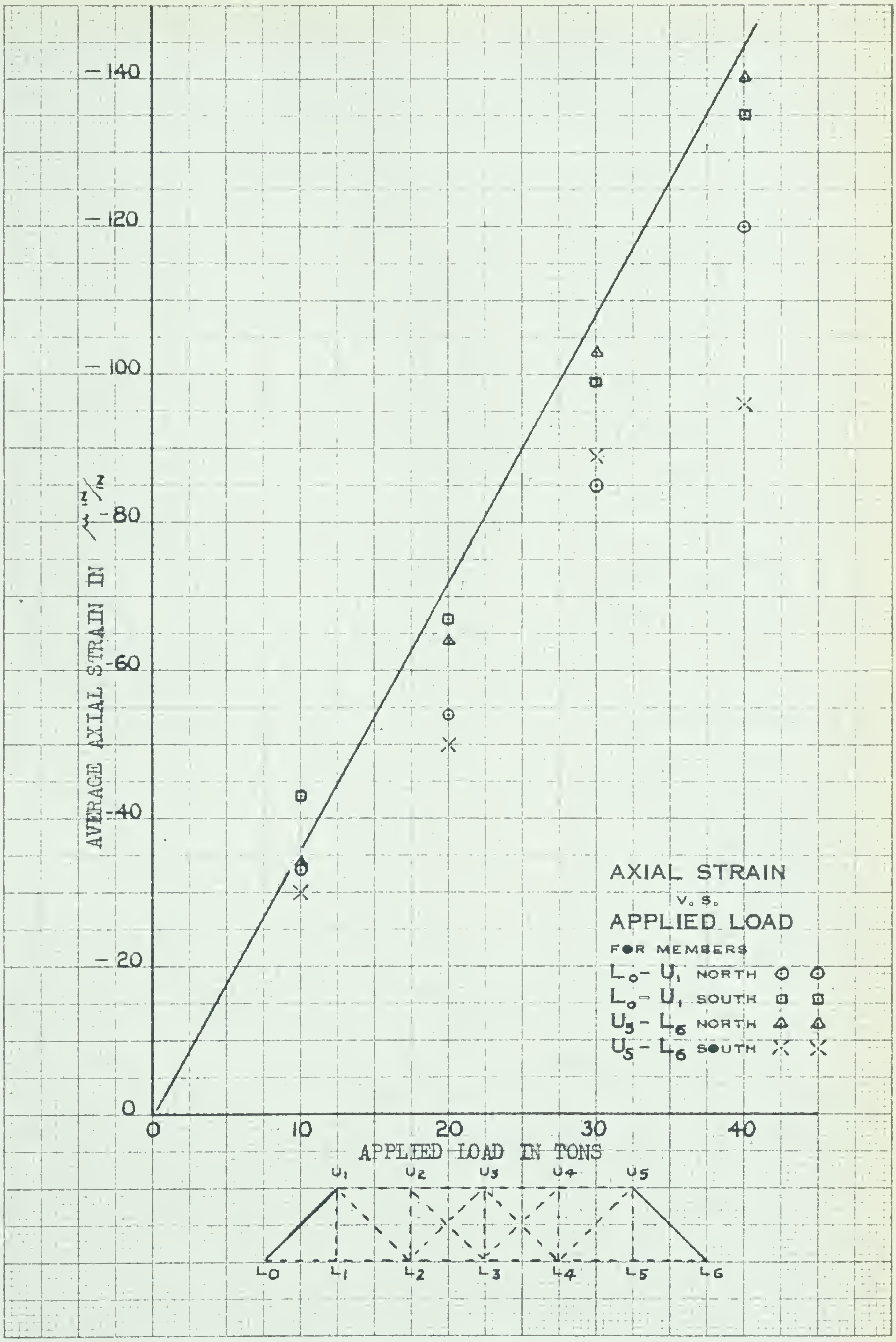
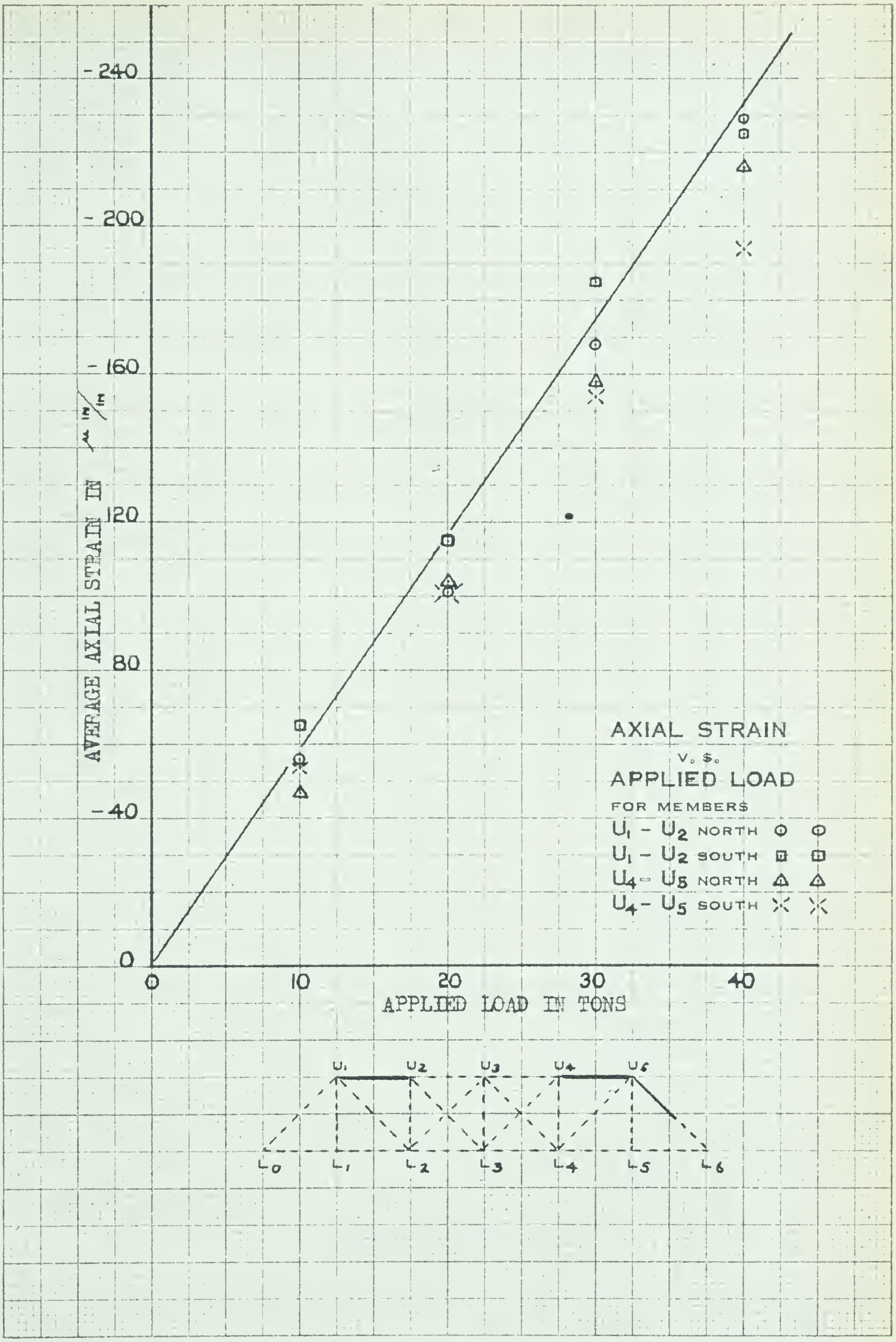

AVERAGE AXIAL STRAIN IN μ in/in
−140
−120
−100
−80
−60
−40
−20
0
0
10
20
30
40
APPLIED LOAD IN TONS
AXIAL STRAIN
V. S.
APPLIED LOAD
FOR MEMBERS
L0 - U1 NORTH
L0 - U1 SOUTH
U5 - L6 NORTH
U5 - L6 SOUTH
U1
U2
U3
U4
U5
L0
L1
L2
L3
L4
L5
L6

Fig. 37



AVERAGE AXIAL STRAIN IN μ in/in
-240
-200
-160
120
80
-40
0
0
10
20
30
40
APPLIED LOAD IN TONS
AXIAL STRAIN
V. S.
APPLIED LOAD
FOR MEMBERS
U1 - U2 NORTH
U1 - U2 SOUTH
U4 - U5 NORTH
U4 - U5 SOUTH
U1
U2
U3
U4
U5
L0
L1
L2
L3
L4
L5
L6

Fig. 38

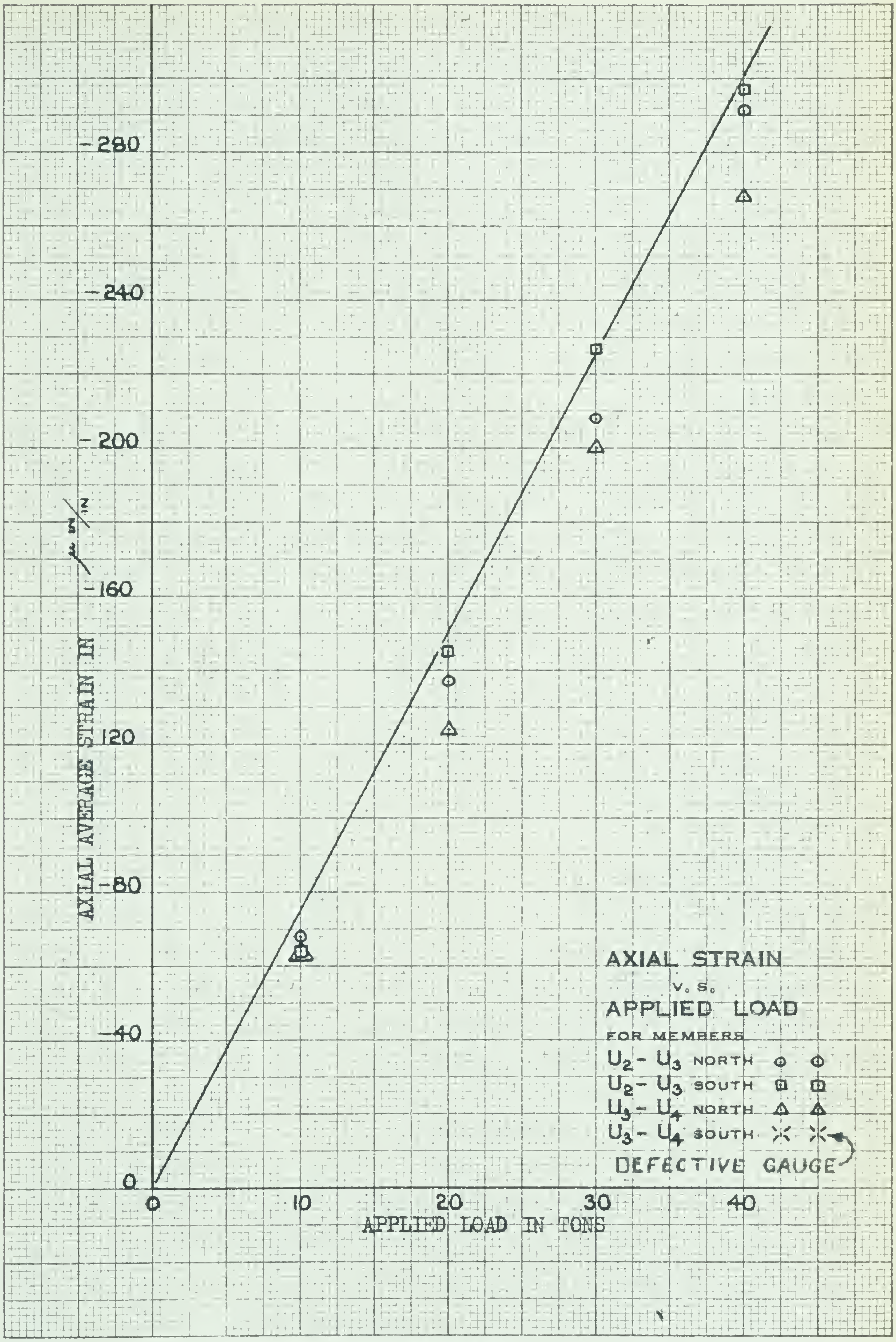

-280
-240
-200
-160
120
-80
-40
0
0
10
20
30
40
AXIAL AVERAGE STRAIN IN μ in/in
APPLIED LOAD IN TONS
AXIAL STRAIN
V. S.
APPLIED LOAD
FOR MEMBERS
$U_2 - U_3$ NORTH
$U_2 - U_3$ SOUTH
$U_3 - U_4$ NORTH
$U_3 - U_4$ SOUTH
DEFECTIVE GAUGE

Fig. 39

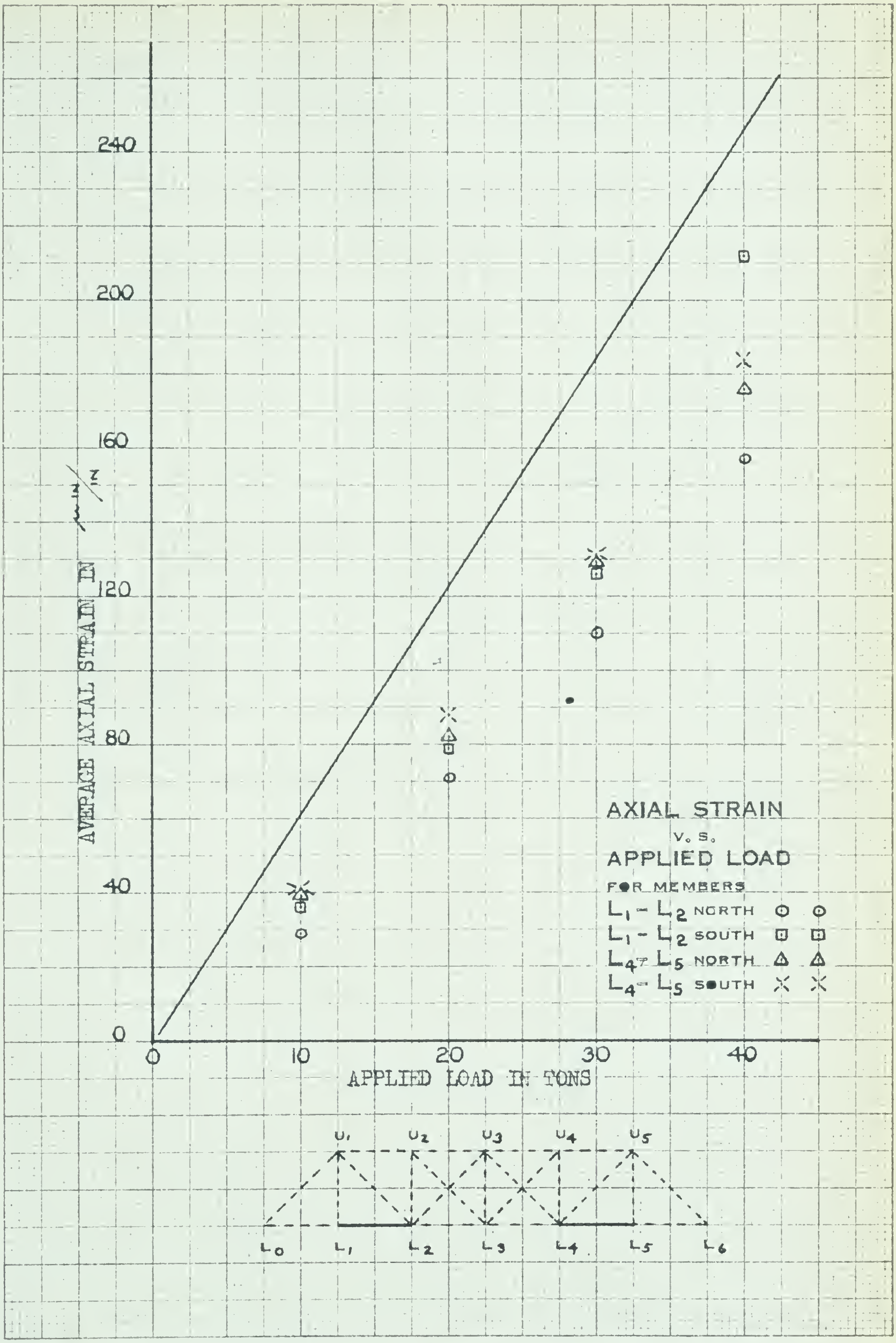
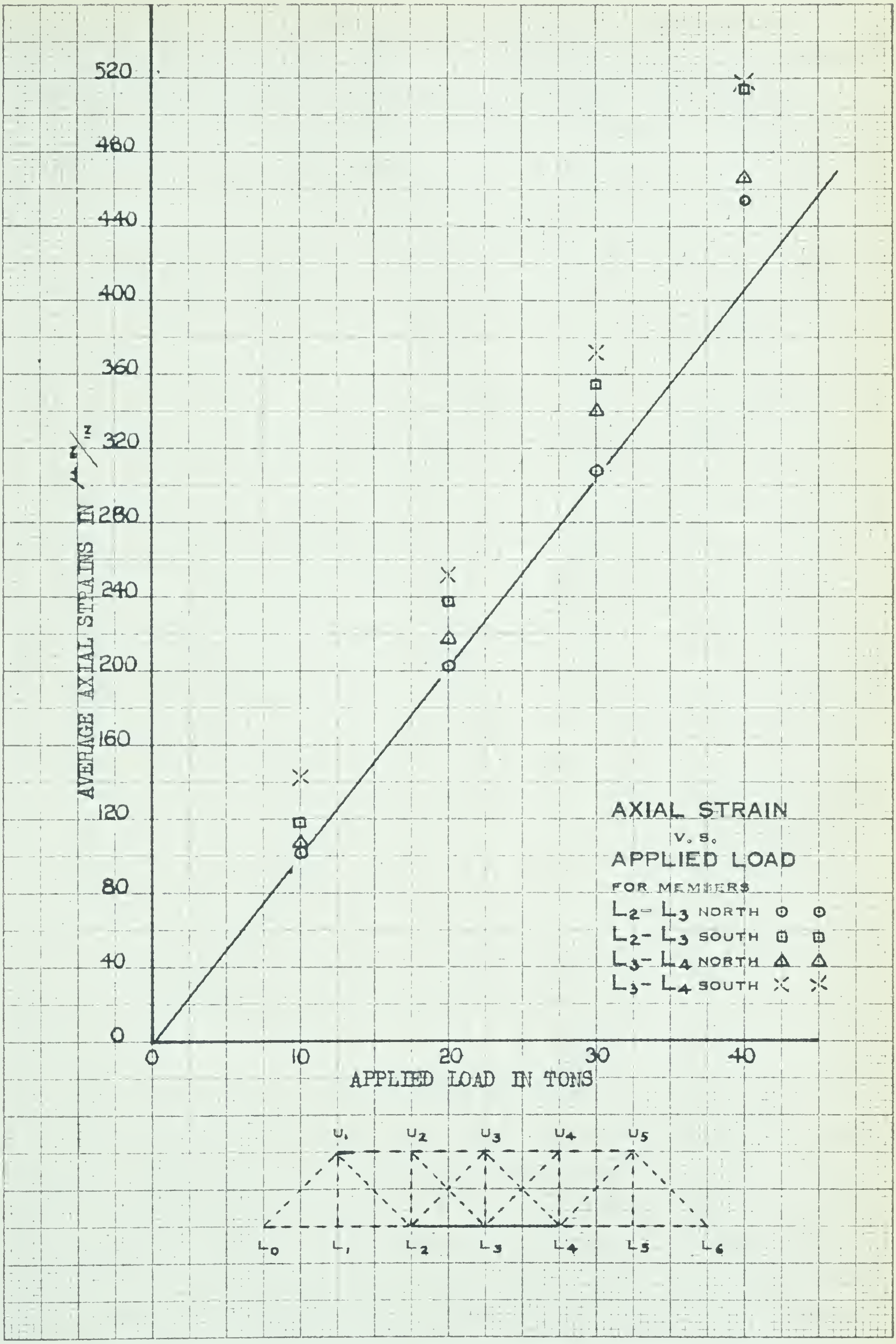

240
200
160
120
80
40
0
AVERAGE AXIAL STRAIN IN μ IN/IN
0
10
20
30
40
APPLIED LOAD IN TONS
AXIAL STRAIN
V. S.
APPLIED LOAD
FOR MEMBERS
L1 - L2 NORTH
L1 - L2 SOUTH
L4 - L5 NORTH
L4 - L5 SOUTH
U1
U2
U3
U4
U5
L0
L1
L2
L3
L4
L5
L6

Fig. 40



520
480
440
400
360
320
280
240
200
160
120
80
40
0
AVERAGE AXIAL STRAINS IN
0
10
20
30
40
APPLIED LOAD IN TONS
AXIAL STRAIN
V. S.
APPLIED LOAD
FOR MEMBERS
L2- L3 NORTH
L2- L3 SOUTH
L3- L4 NORTH
L3- L4 SOUTH
U1
U2
U3
U4
U5
L0
L1
L2
L3
L4
L5
L6

Fig. 41

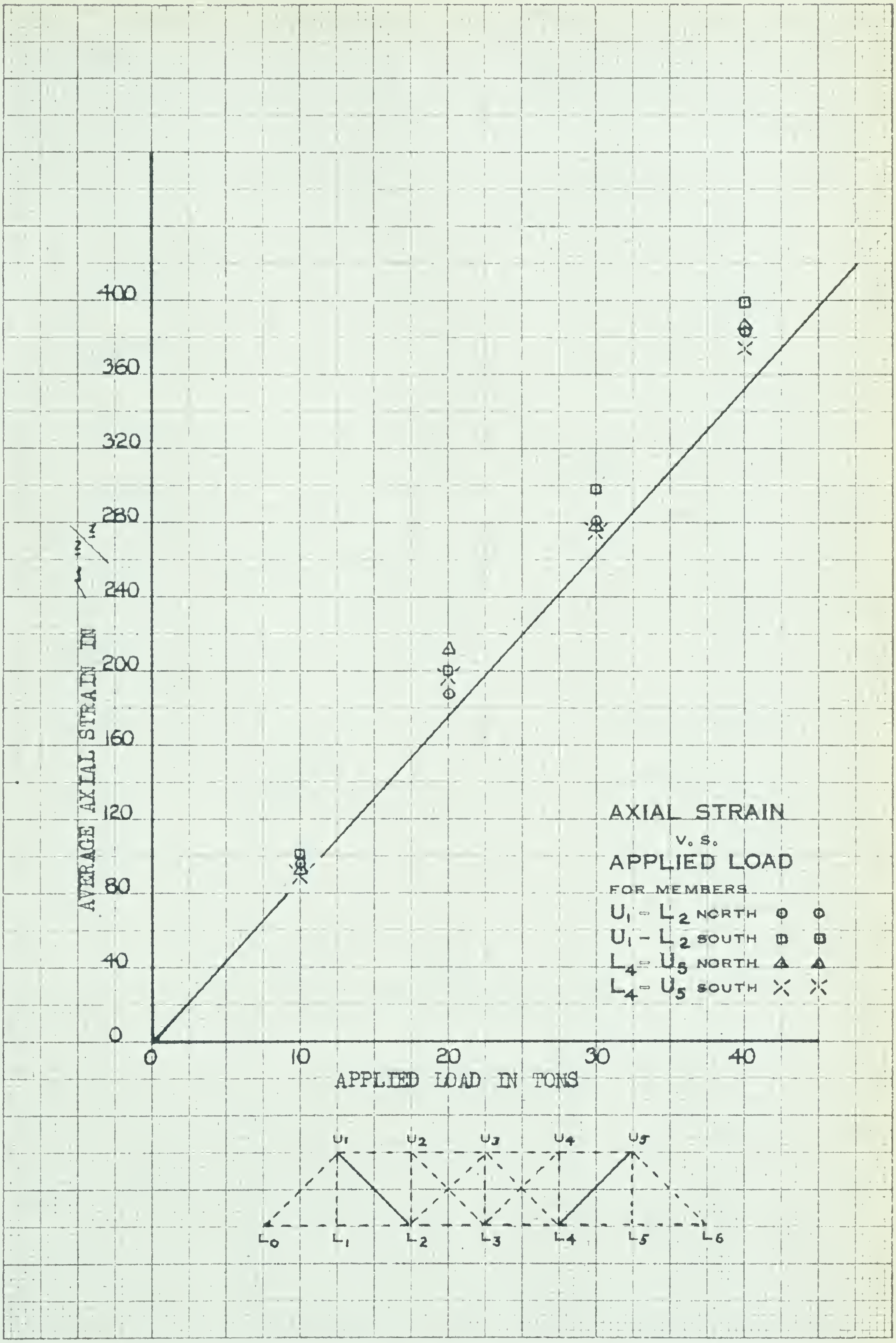

AXIAL STRAIN
V. S.
APPLIED LOAD
FOR MEMBERS
$U_1 - L_2$ NORTH
$U_1 - L_2$ SOUTH
$L_4 - U_5$ NORTH
$L_4 - U_5$ SOUTH
AVERAGE AXIAL STRAIN IN
0
40
80
120
160
200
240
280
320
360
400
0
10
20
30
40
APPLIED LOAD IN TONS
U_1
U_2
U_3
U_4
U_5
L_0
L_1
L_2
L_3
L_4
L_5
L_6

Fig. 42

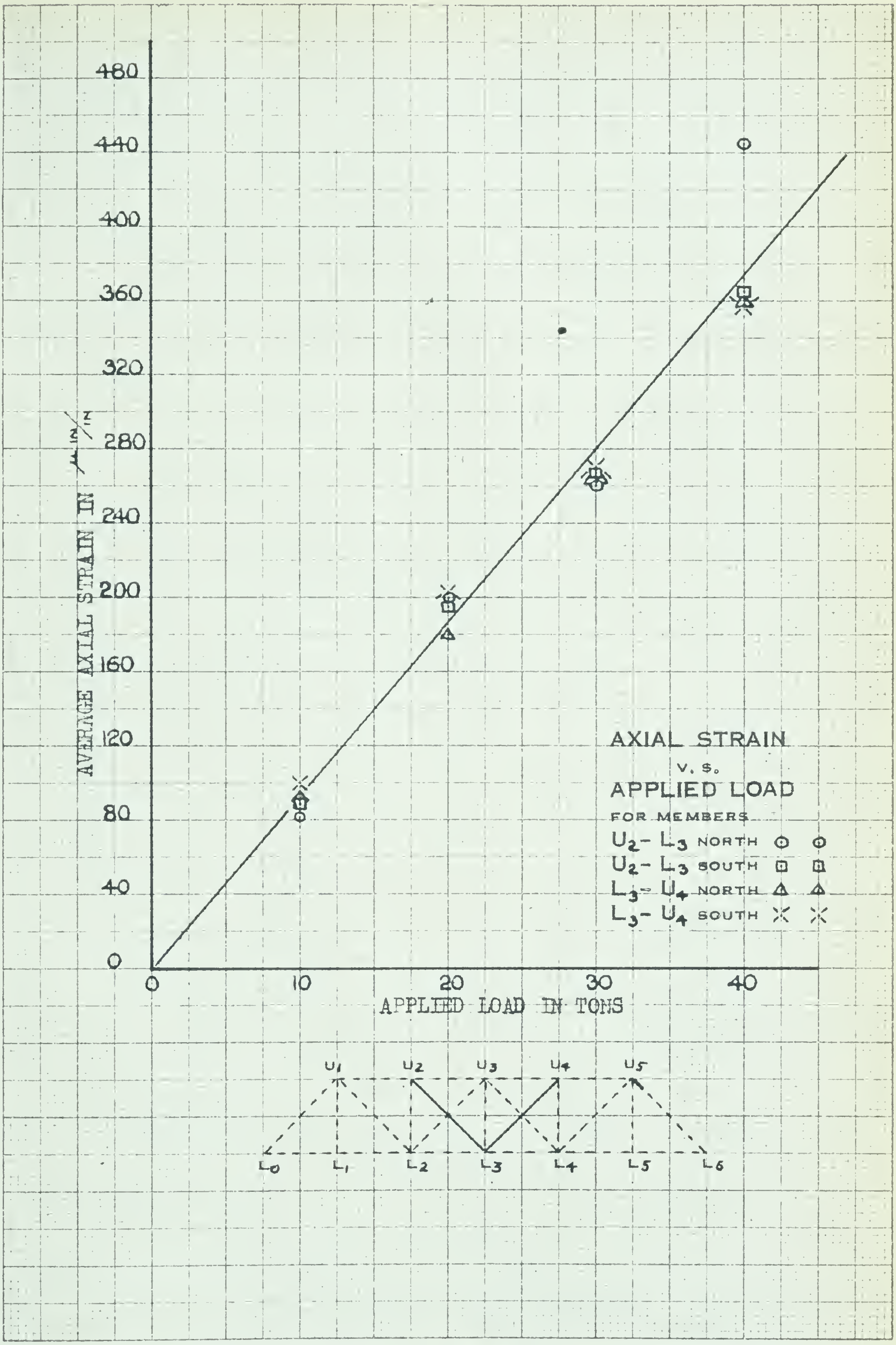
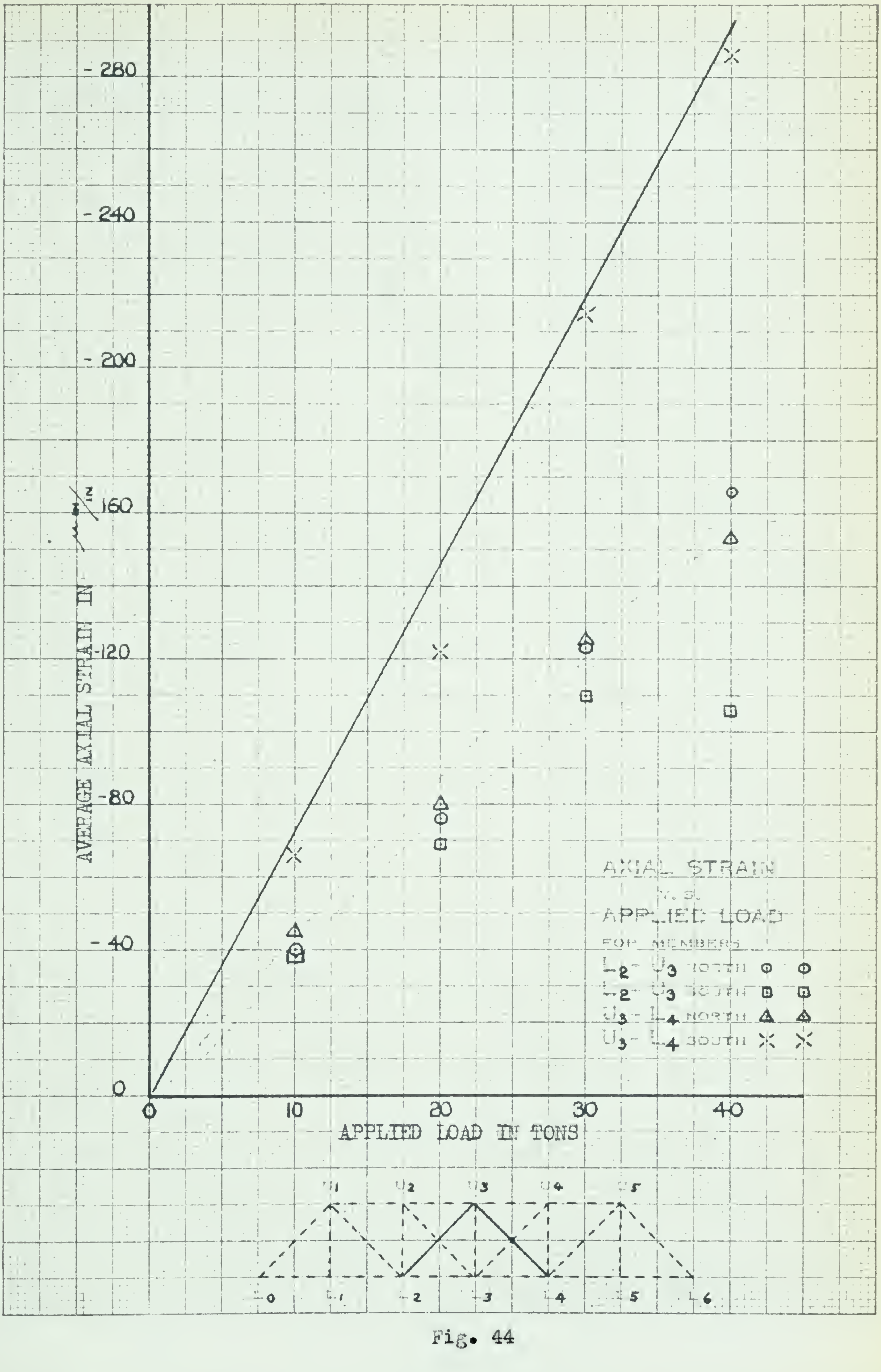

AVERAGE AXIAL STRAIN IN μ IN/IN
480
440
400
360
320
280
240
200
160
120
80
40
0
0
10
20
30
40
APPLIED LOAD IN TONS
AXIAL STRAIN
V. S.
APPLIED LOAD
FOR MEMBERS
U2 - L3 NORTH
U2 - L3 SOUTH
L3 - U4 NORTH
L3 - U4 SOUTH
U1
U2
U3
U4
U5
L0
L1
L2
L3
L4
L5
L6

Fig. 43



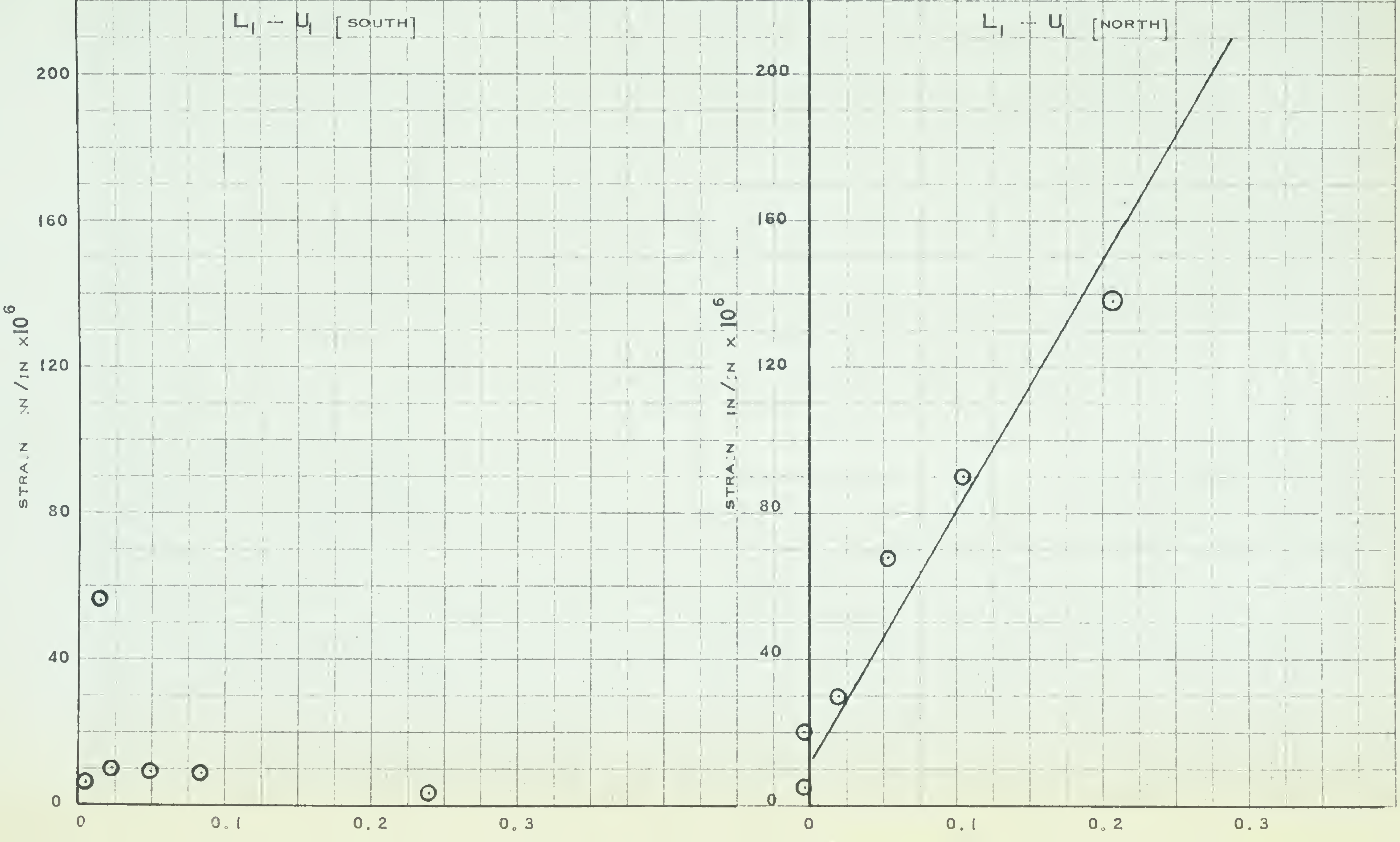
AVERAGE AXIAL STRAIN IN μ in/in
-280
-240
-200
-160
-120
-80
-40
0
0
10
20
30
40
APPLIED LOAD IN TONS
AXIAL STRAIN
V. S.
APPLIED LOAD
FOR MEMBERS
L2 - U3 NORTH
L2 - U3 SOUTH
U3 - L4 NORTH
U3 - L4 SOUTH
U1
U2
U3
U4
U5
L0
L1
L2
L3
L4
L5
L6

Fig. 44

PLOT OF FLEXURAL STRAIN ON THE WEB VERTICAL

V. S.

DEFLECTION OF THE TOP OF THE VERTICAL



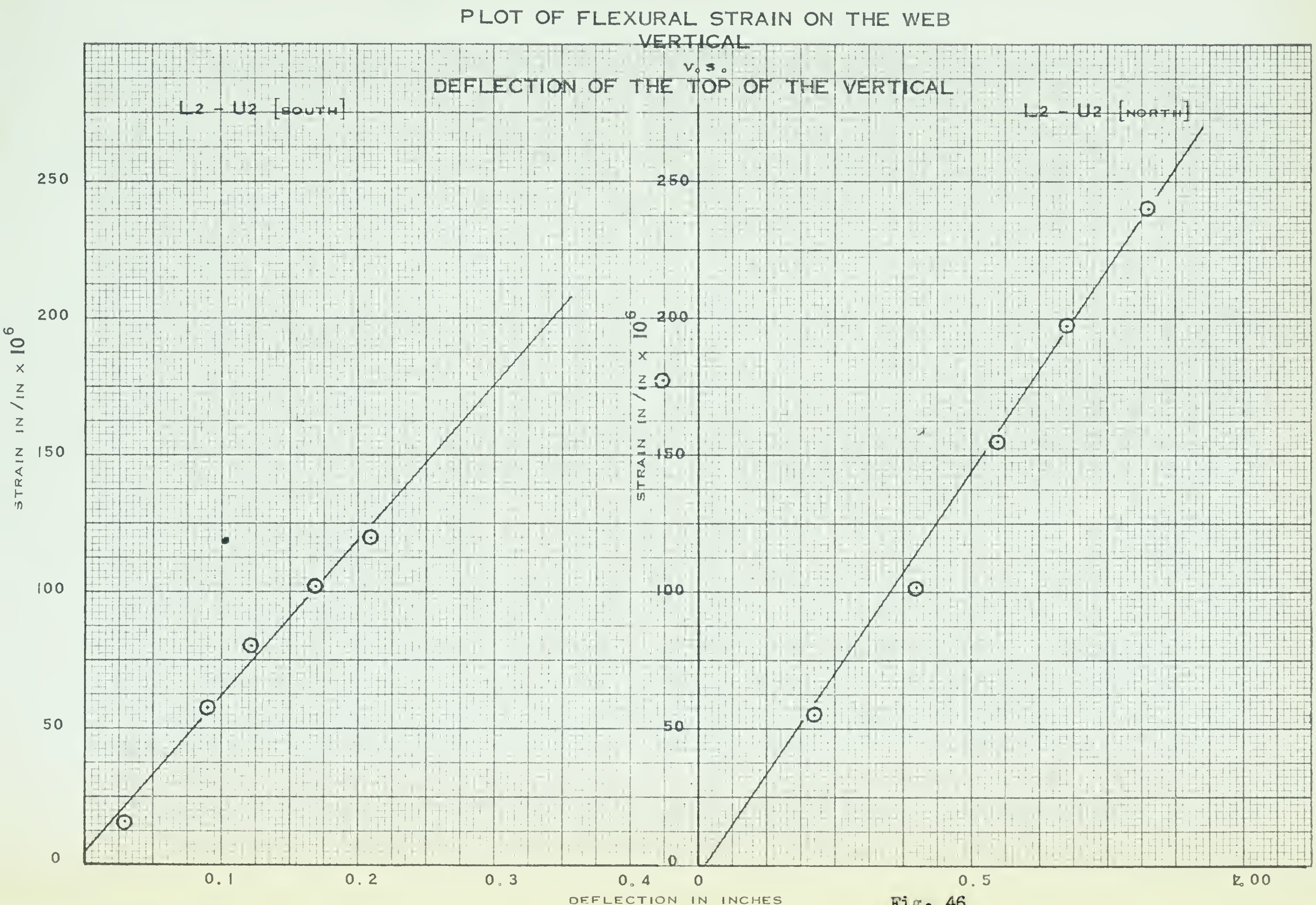

PLOT OF FLEXURAL STRAIN ON THE WEB VERTICAL
V.S.
DEFLECTION OF THE TOP OF THE VERTICAL
L2 - U2 [SOUTH]
L2 - U2 [NORTH]
STRAIN IN / IN × 10^6
0
50
100
150
200
250
0.1
0.2
0.3
0.4
0
0.5
1.00
DEFLECTION IN INCHES

Fig. 46

PLOT OF FLEXURAL STRAIN ON THE WEB VERTICAL

V. S.

DEFLECTION OF THE TOP OF THE VERTICAL

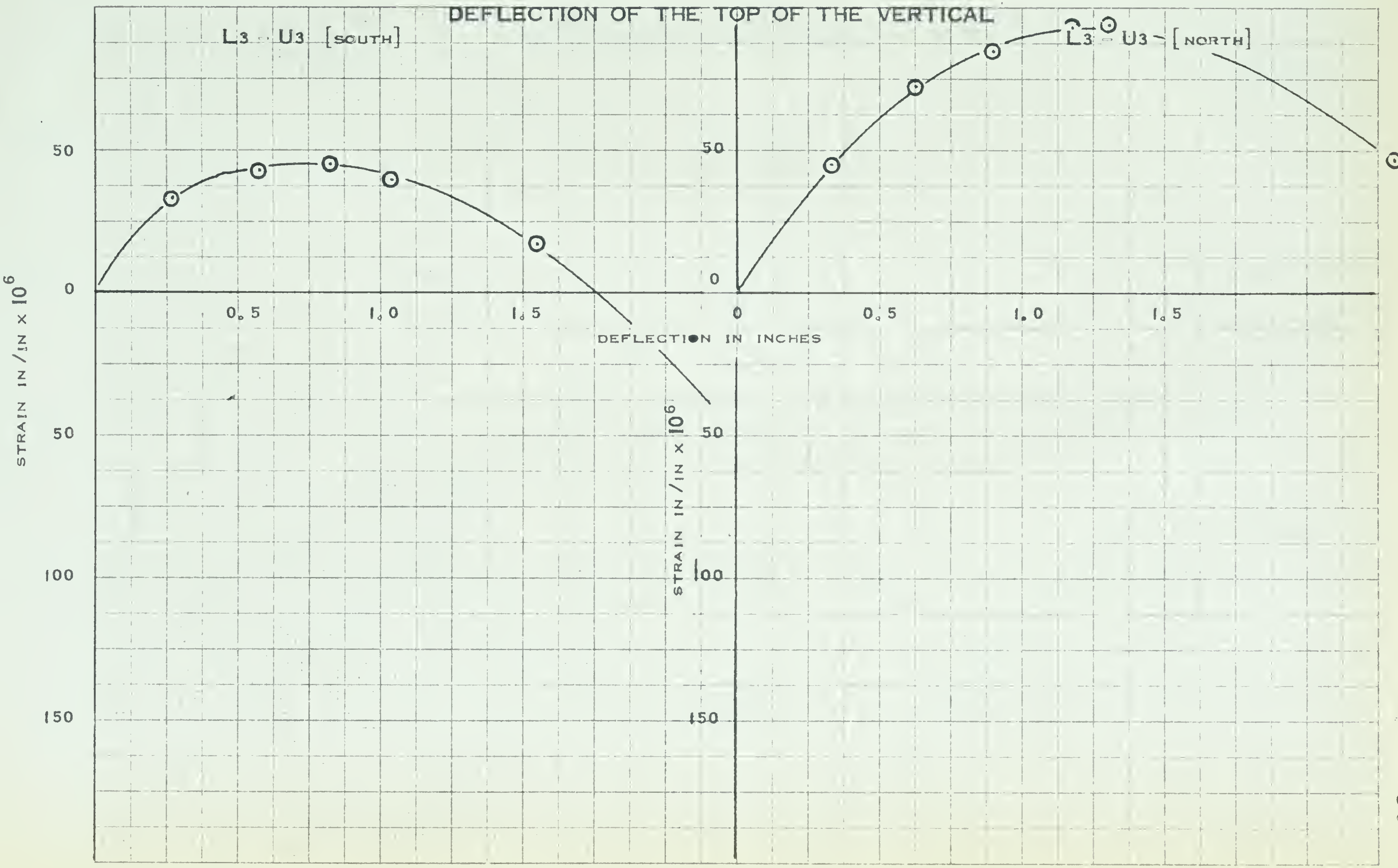


Fig. 47

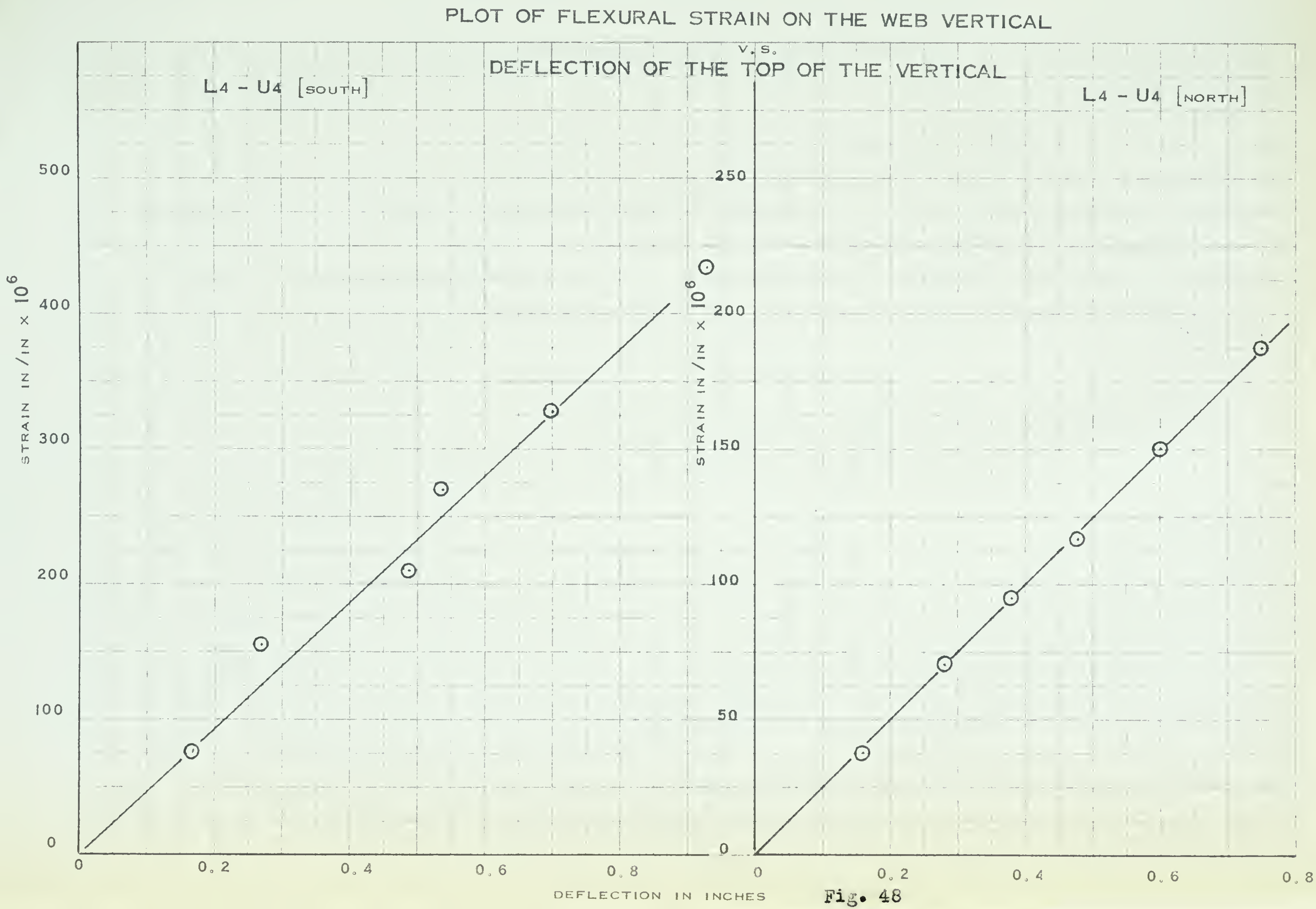

PLOT OF FLEXURAL STRAIN ON THE WEB VERTICAL
V. S.
DEFLECTION OF THE TOP OF THE VERTICAL
L4 - U4 [SOUTH]
L4 - U4 [NORTH]
STRAIN IN /IN × 10^6
0
100
200
300
400
500
50
150
250
0.2
0.4
0.6
0.8
DEFLECTION IN INCHES

Fig. 48

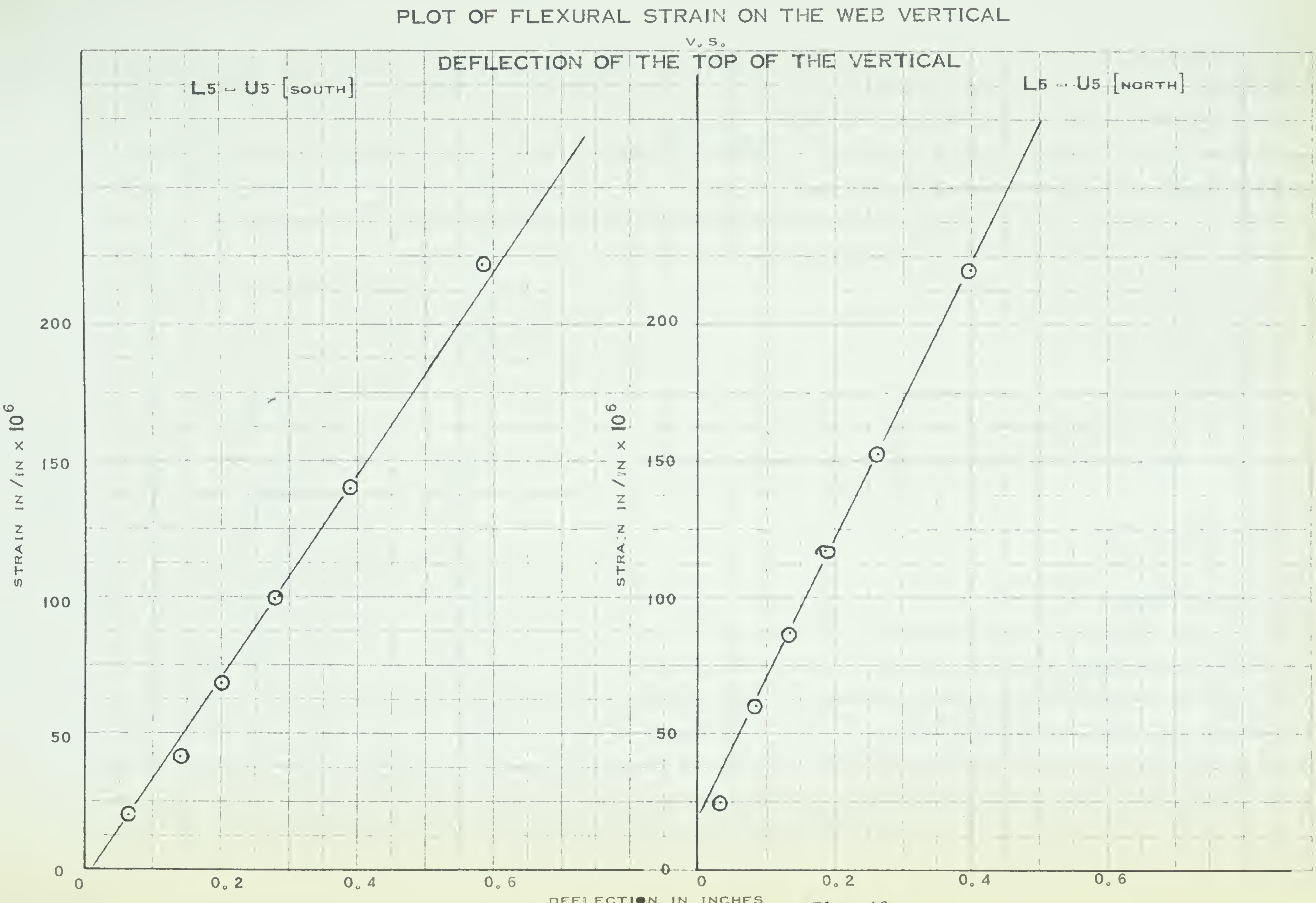

PLOT OF FLEXURAL STRAIN ON THE WEB VERTICAL
V. S.
DEFLECTION OF THE TOP OF THE VERTICAL
L5 - U5 [SOUTH]
L5 - U5 [NORTH]
STRAIN IN / IN x 10^6
0
50
100
150
200
0
0.2
0.4
0.6
DEFLECTION IN INCHES

Fig. 49

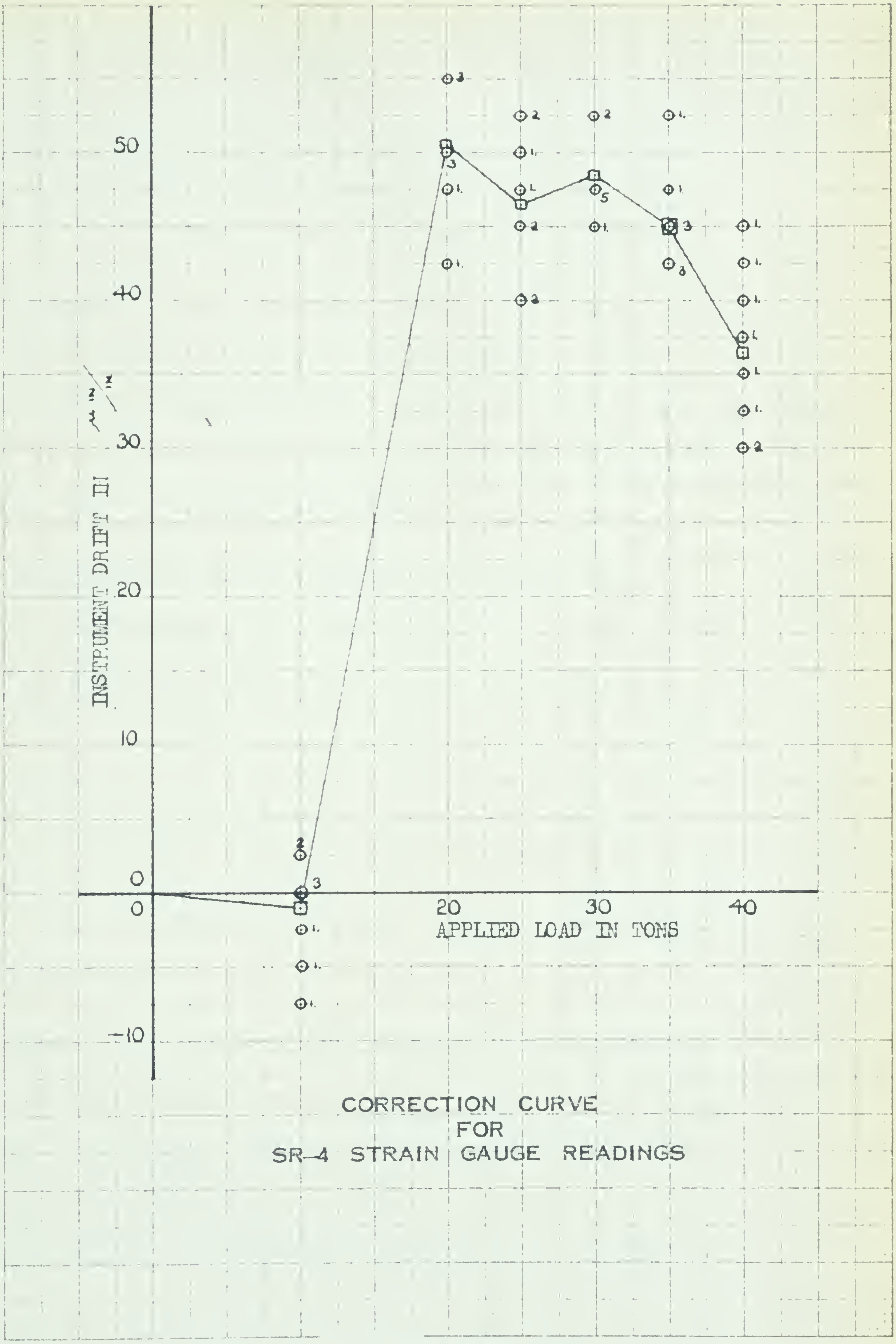

INSTRUMENT DRIFT III
50
40
30
20
10
0
-10
0
20
30
40
APPLIED LOAD IN TONS
CORRECTION CURVE
FOR
SR-4 STRAIN GAUGE READINGS

Fig. 50

APPENDIX III

PRELIMINARY TEST

TEST #1

OBJECT - To determine the location of and the minimum number of "SR-4" strain gauges required on single angle members such that the average axial strain could be obtained.

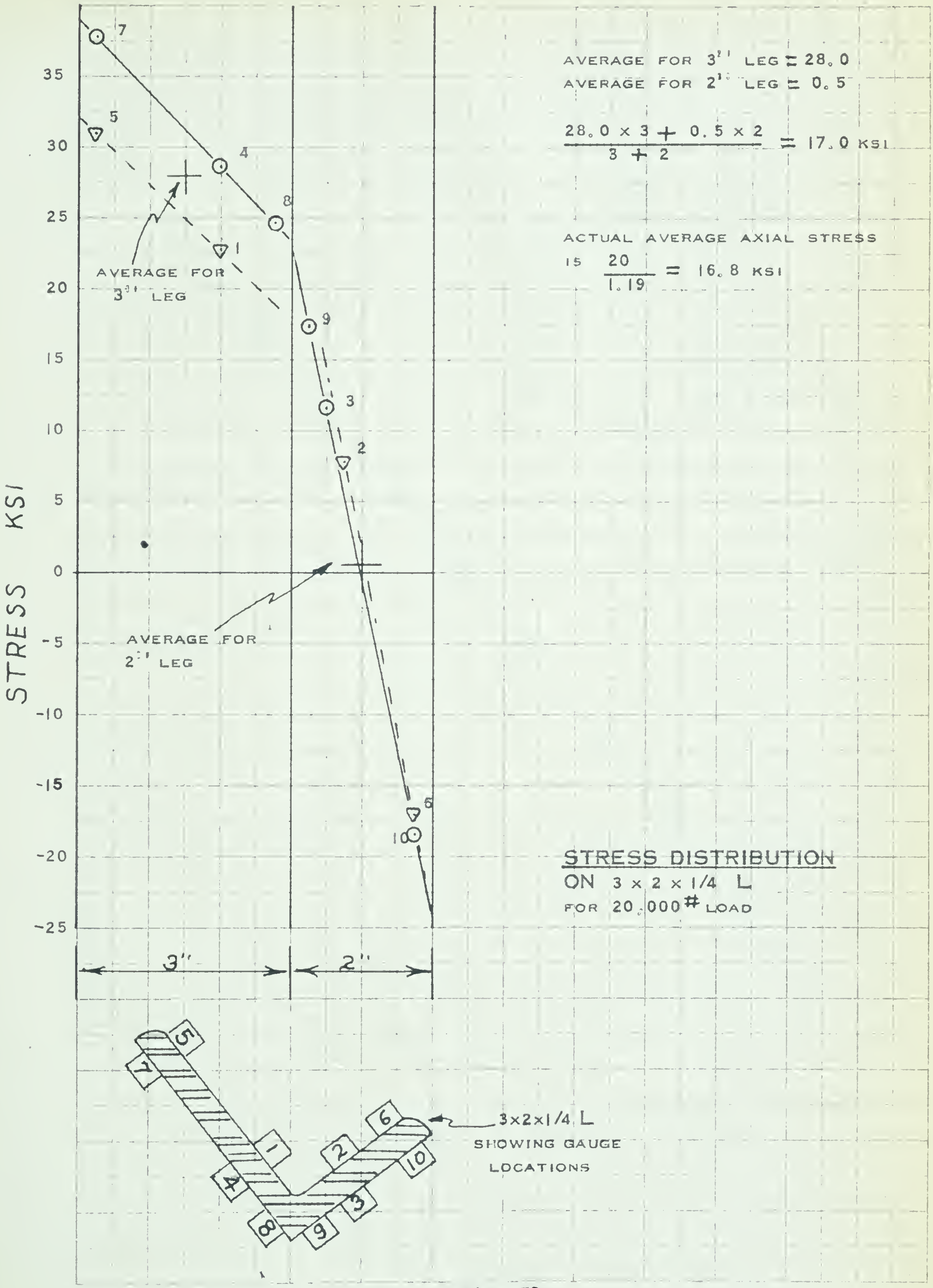
SPECIMEN - A 2 x 3 x 1/4 angle 30" long was used for this purpose. In order to be able to clamp the specimen in the Baldwin testing machine as well as to be able to apply a load on the rivet gauge line two 2 x 1/2 inch flats were welded onto the ends of the 3 inch leg of the angle.

PROCEDURE - A total of 14 "SR-4" strain gauges were attached to the specimen. They were located in pairs as follows, on the extreme fiber of each leg or toe, on the neutral axis of each leg, on the heel of the angle and in the middle of each leg. Gauges one to ten are shown on Fig. 51. All gauges were located as close as possible in the middle of the specimen. The specimen was then placed in the Baldwin testing machine and loaded in 2 kip increments to 20 kips. The strains were recorded for each increment.

RESULTS - The results indicate that only four gauges are necessary to obtain the average strain in a single angle member. These four gauges are the two pairs located at the middle of each leg. By taking the average strain of each leg (i.e. by averaging each pair of gauges), and multiplying by the width of

the corresponding leg plus the similar strain multiplied by the average strain of the other leg and then dividing by the sum of the lengths of the two legs, the end result is close to the average axial strain in the member. The results of these separate runs are plotted along with the theoretical average strains on Fig. 52.

Another significant point that was observed from this test, one which is not directly concerned with the object, is the stress distribution across the angle section. As can be seen on Fig. 52, the average axial strain was 16.8 ksi whereas the extreme fibre stress varied from +39 ksi on the connected leg to -24 ksi on the other leg.



AVERAGE FOR 3'' LEG = 28.0
AVERAGE FOR 2'' LEG = 0.5
28.0 x 3 + 0.5 x 2 / 3 + 2 = 17.0 KSI
ACTUAL AVERAGE AXIAL STRESS
IS 20 / 1.19 = 16.8 KSI
AVERAGE FOR 3'' LEG
AVERAGE FOR 2'' LEG
STRESS KSI
STRESS DISTRIBUTION
ON 3 x 2 x 1/4 L
FOR 20,000# LOAD
3''
2''
3x2x1/4 L
SHOWING GAUGE
LOCATIONS

Fig. 51

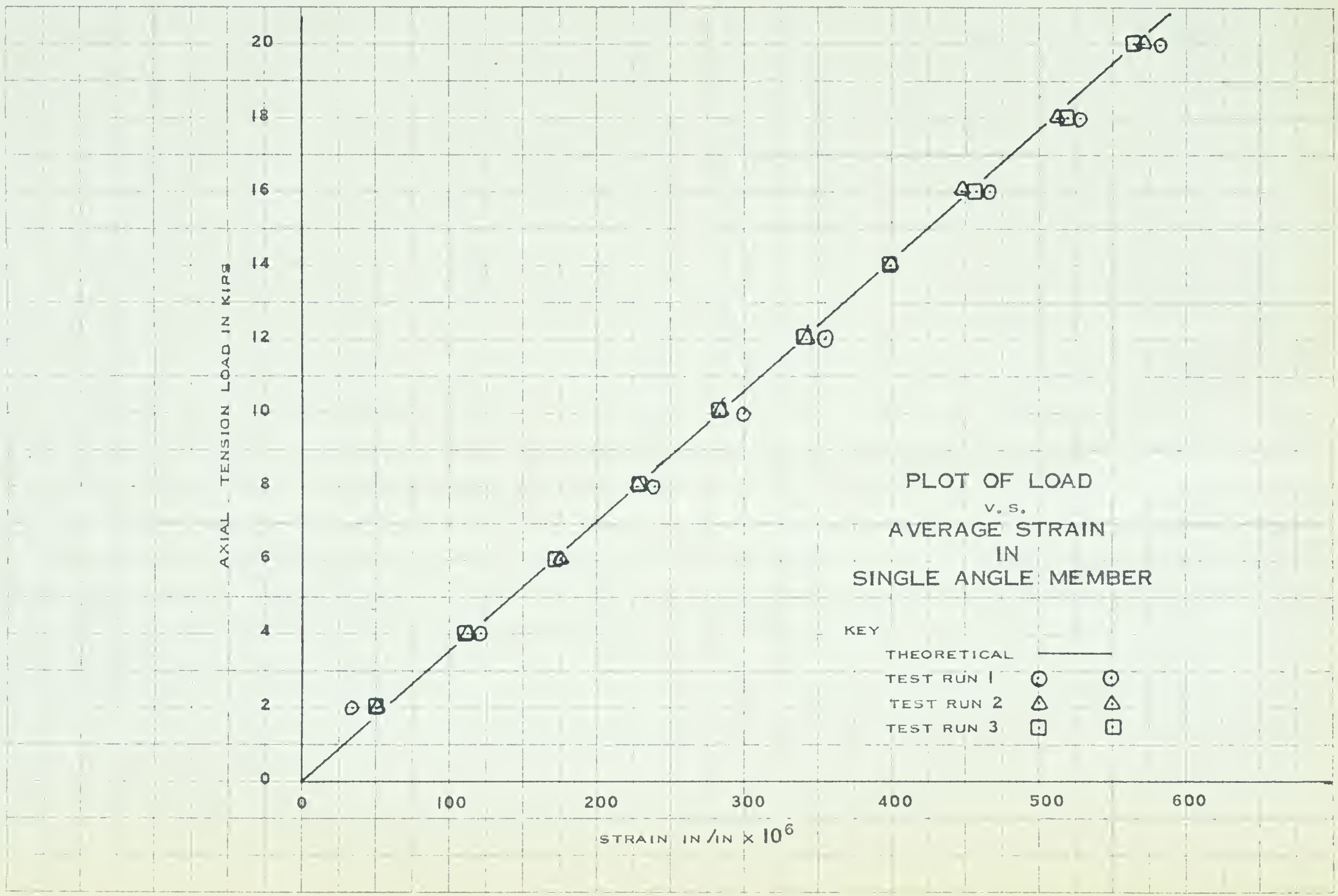

PLOT OF LOAD
V. S.
AVERAGE STRAIN
IN
SINGLE ANGLE MEMBER
KEY
THEORETICAL
TEST RUN 1
TEST RUN 2
TEST RUN 3
AXIAL TENSION LOAD IN KIPS
STRAIN IN /IN x 10^6
0
2
4
6
8
10
12
14
16
18
20
0
100
200
300
400
500
600

Fig. 52

PRELIMINARY TEST

TEST #2

OBJECT - To check out the proper functions of the loading apparatus and all the instrumentation of the pony-truss bridge.

PROCEDURE - The pony-truss bridge was loaded several times up to 20 tons in five ton increments and all the instruments were read and recorded.

OBSERVATIONS -

1. Loading apparatus.
The loading apparatus functioned very well, and no difficulties were encountered.

2. Vertical Deflection Instrumentation
The instrument used for measuring the vertical deflections operated satisfactorily.

3. Angle changes between the floor members and the vertical member.

This apparatus functioned satisfactorily with the exception of those dials which were located too close to the wood stringers. It was observed that the wood stringers moved outwards when a load was applied on the bridge, and came in contact with the dials. This prevented the proper functioning of the dials. To eliminate this some of the wood was cut from the stringers to give ample

clearance for these dials.

4. "SR-4" Strain Gauges and Strain Indicators.

Most difficulty was experienced with this instrumentation. One problem which has been already mentioned was that direct sunlight affected the strain gauges. This, however, was solved by carrying out the tests at night. Secondly all three available indicators produced erratic results at times. The causes for these erratic results were finally discovered. Two of the indicators were not in proper operating condition and the third one which was a late model (transistor type) would produce good results only after it had been switched on for 20 to 30 minutes.

Another problem which was not realized during the preliminary tests but after the actual failure test is that battery operated indicators are not good for tests of long duration, if readings are taken continuously. The explanation for this is that if the voltage of the battery drops, the datum of the indicator changes. That is, strain indicator readings taken with a new battery and then with an old battery, on a specimen under identical physical conditions for both readings, may show a variation up to seventy micro-inches. This shift in the datum may be taken into account by taking readings on unstrained gauges at regular intervals.

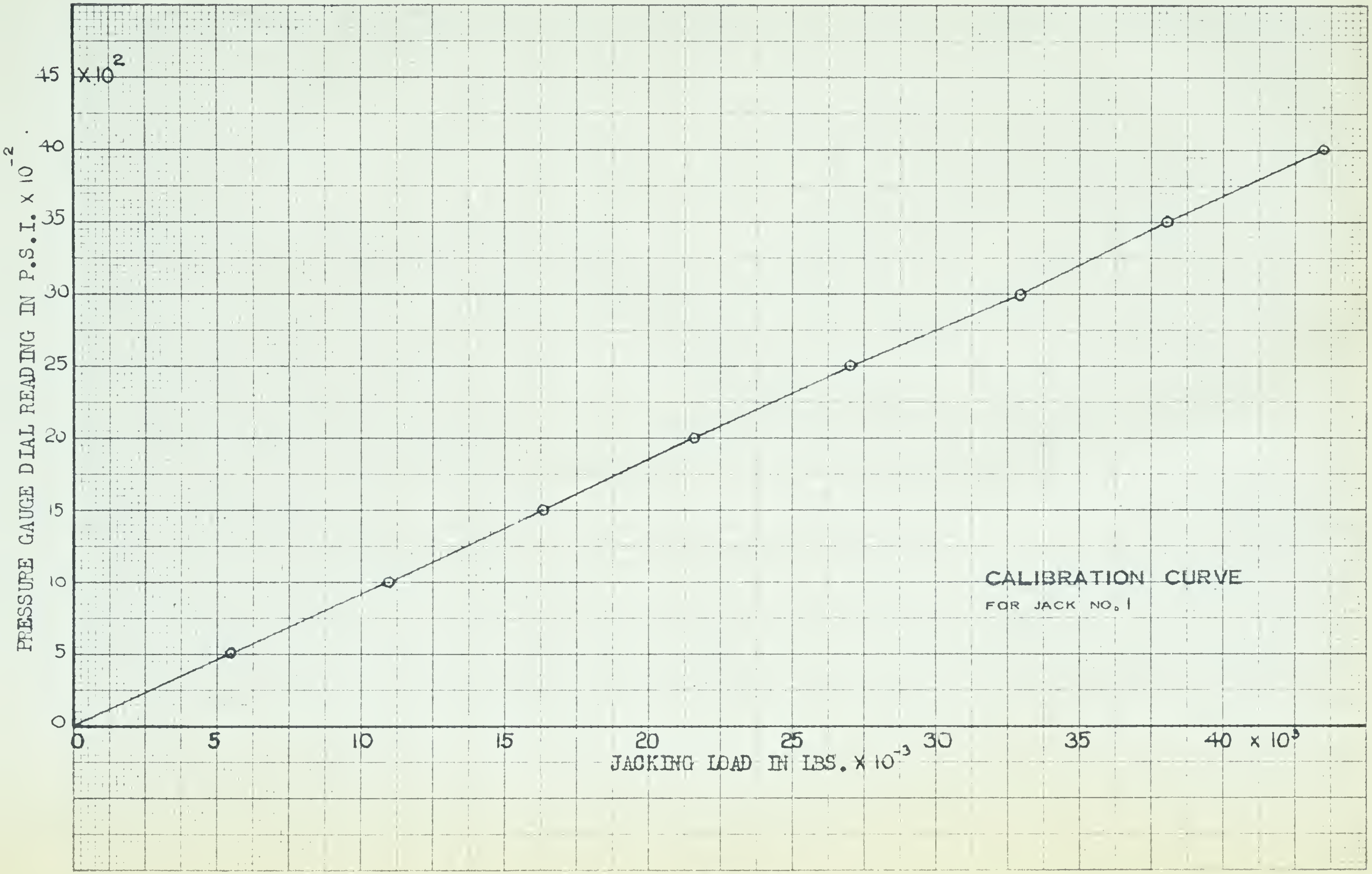

CALIBRATION CURVE
FOR JACK NO. 1
PRESSURE GAUGE DIAL READING IN P.S.I. x 10^-2
X 10^2
0
5
10
15
20
25
30
35
40
45
JACKING LOAD IN LBS. X 10^-3
0
5
10
15
20
25
30
35
40 x 10^3

Fig. 53

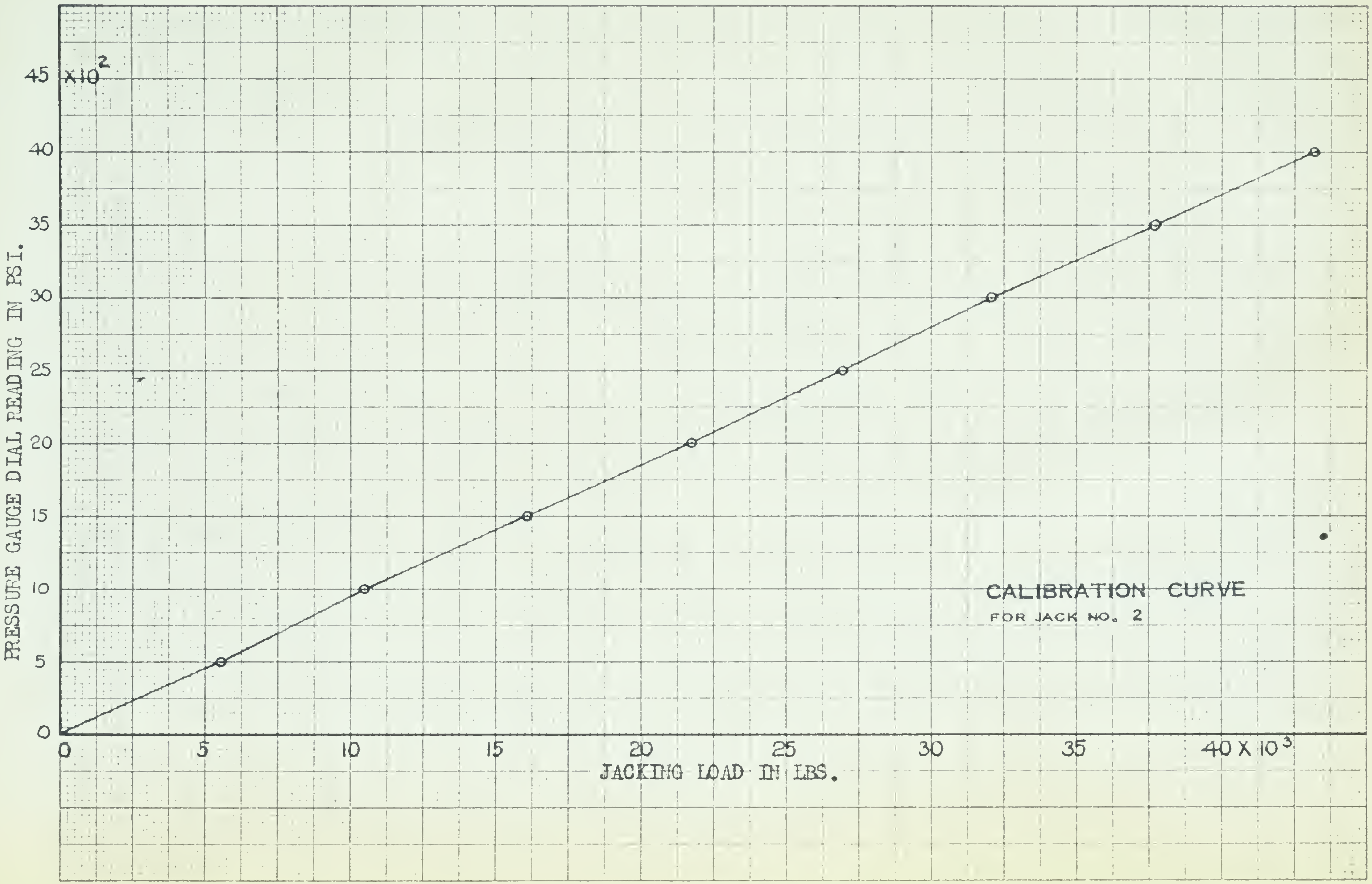

45 x10^2
40
35
30
25
20
15
10
5
0
PRESSURE GAUGE DIAL READING IN PSI.
0
5
10
15
20
25
30
35
40 X 10^3
JACKING LOAD IN LBS.
CALIBRATION CURVE
FOR JACK NO. 2

Fig. 54